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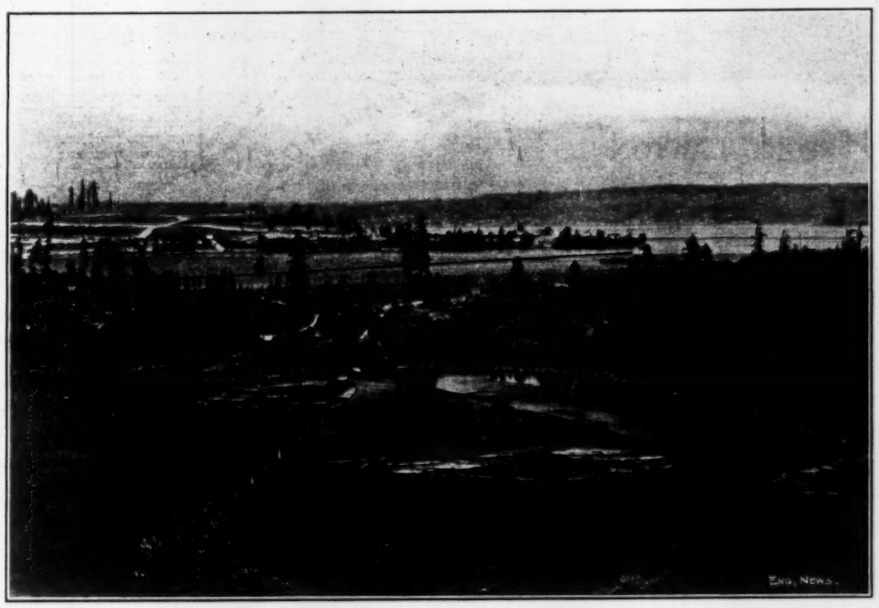
The Proposed Lake Washington Canal, Seattle, Wash.: A Great Engineering Project.

By ARCHIBALD O. POWELL,* M. Am. Soc. C. E.

The following article has been prepared with the double purpose of directing attention to an important public work in the western section of our country and of illustrating one of the chief difficulties which retard the progress of nearly all important engineering enterprises. I refer to the artificial obstacles growing out of the occupancy of the soil by the works and operations of man. The engineer can generally get along with Nature fairly well, but man often proves too much for him. In the example here considered Nature is perfectly tractable and accommodating and the obstacles that have thus far successfully barred the way are of a purely artificial character.

On May 20, 1908, the Secretary of War transmitted to Congress, with the favorable endorsements of the Chief of Engineers and the Board of Engineers for Rivers and Harbors, an exhaustive report by Major (now Lieutenant-Colonel) H. M. Chittenden, M. Am. Soc. C. E., upon a project to construct a ship canal connecting the tidal waters of Puget Sound, via Salmon Bay, with the fresh-water lakes, Union and Washington, lying within or contiguous to the city of Seattle. This is what is popularly known as the Lake Washington Canal—a project as old as the city itself.† It is the outgrowth of topographic conditions unique in the history of the world's great cities, in which an extensive salt-water harbor can, with comparatively little effort, be more than quadrupled in area and three times quadrupled in shore line by a fresh water-non-freezing, non-tidal harbor of ample depth for the heaviest shipping. The main portion of Lake Union has 40 ft. of water, while Lake Washington, except in the shallow part of Union Bay, varies from 40 ft. near the shore to 200 ft. in the deepest holes in the center of the lake. One of the chief features of the lakes favorable to harbor and waterway purposes is the absence of ice. Although Puget Sound is located in the middle of the North temperate zone, the winters are mild and not markedly different in temperature from the springs and falls. Traffic on the lakes is not interrupted by ice or stress of weather. The accompanying map (Fig. 2) exhibits the relation of Elliott Bay (Seattle's harbor) and the small tidal slough known as Salmon Bay to Lakes Union and Washington. The actual land separation aggregates about a mile and a half in length, while its elevation above extreme high tide is only 7 ft. for Lake Union and about 15 ft. for the larger lake. The two lakes together have about 24,000 acres of navigable water deep enough to float the largest vessels.



LAKE UNION, UNION BAY AND LAKE WASHINGTON, SEATTLE, WASH.

(In near foreground is the upper end of Lake Union. Neck of land just back is the Portage. Union Bay back of Portage. Next is Laurelhurst Peninsula, a beautiful new residence district. Still further is the main body of Lake Washington with the East shore in the distance. In clear weather the snow-capped Cascades define the horizon. In the center of the picture is the old Government right-of-way across the Portage. A cut was made through this many years ago and small wooden locks were put in but have since been removed. Coal used to be brought to the city via the Portage from the Renton mine at the south end of Lake Washington. The channel is now used only for sluicing logs through for the mills on Lake Union. The black line shows the newly adopted route of the canal through the Portage and the shoal water of Union Bay. The University is just to the left outside the picture.)

From the very birth of the city the advantage of navigable access to these two lakes has strongly appealed to its citizens, while government officials from quite as far back have been impressed with its great possibilities to the Navy and the commerce of Puget Sound. In fact, the project has always seemed to be one of those things which Nature has fairly thrown in man's way as an opportunity not to be ignored.

The early proposition to locate the Puget Sound Navy Yard upon the shore of these lakes was rejected for the reason (1) that when the selection of a site was under consideration the lakes were inaccessible, and (2), and more important, that it was not considered advisable to locate the yard behind a lock to which a mishap of any kind might prove a disaster of the first order. Although this danger was probably much exaggerated and was offset, to some extent, by the complete defensibility of the lock from naval attack, still the ver possibility was considered something that should be avoided, and accordingly a location on tide water was sought and found in the intricate estuary directly across the Sound behind Bainbridge Island, 18 miles distant.

The argument that decided the Navy Yard location does not apply, however, to another very important naval establishment—a waiting basin for vessels laid up temporarily in time of peace. For this purpose a fresh water basin is of great advantage. To quote Colonel Chittenden:

> A ship that is constantly under way fouls slowly and her bottom remains comparatively clean for a considerable time. When lying quietly in salt water her bottom fouls rapidly. Deterioration, resulting from electrolytic action and corrosion, is more rapid with a foul bottom than with one that is clean. In fresh water this action does not take place—at least with anything like its rapidity in salt water. In Puget Sound a vessel should be put into dock and cleaned at least once in nine months if it is to be kept in prime condition. In fresh water it would probably be docked and cleaned before putting it into the basin and not again until it is put into commission. To place the vessels in dock at the Navy Yard, and clean them, costs from $800 to $1,000 for a torpedo boat, to $2,500 to $3,000 for a battleship. From these figures it will be seen what a saving would result with a fresh-water basin if a considerable number of vessels are laid up in ordinary for a great length of time. Besides this saving in cost the hulls would remain in better condition than would be possible to maintain them in salt water.

Admiral W. T. Burwell, now retired, whose record is so closely identified with the building of the Puget Sound Navy Yard, was always a firm believer in the utility of these bodies of fresh water for a naval waiting basin. In one of his official reports he thus refers to the matter:

> The vast extent of Lake Washington provides an

*Central Building, Seattle, Wash.

[†See our issues of March 15 and July 12, 1894; Jan. 24, 1895, Dec. 31, 1896 (in "Notes of a Transcontinental Trip"), and May 27, 1897. A so-called South Canal, projected by the Lake Washington Waterway Co. was also described in some of the articles referred to.—Ed.]

abundance of room for any number of vessels, which can lie freely at anchor, whatever be the extent of the Government property ashore; torpedo boats can exercise under steam without even going outside; and the greatest freedom and the widest opportunity can be had in development. In fact, this magnificent lake is not comparable with a basin nor even with a river; but is a large, fresh-water harbor, the like of which does not exist elsewhere. . . . The use of a fresh-water basin is to lay up naval vessels in time of peace; not as a refuge to run them into in time of war. . . .

Besides this, sight must not be lost of the fact that the use of a fresh-water basin is not confined to war vessels of a large size; but colliers, transports, torpedo boats, repair ships, water ships, supply ships, and auxiliary vessels of all kinds should have its advantages.

into the deep water that crowds up to the very shore line.

Moreover, the frontage of Elliott Bay is too limited for the future needs of such a port as Seattle is bound to become. The entire perimeter of Elliott Bay, from the West Point Light to Duwamish Head, is less than ten miles, and of this fully 50% is not available in a practical sense on account of exposed position or the precipitous character of the shores. Add to this the fact that about 60% of the available balance is

timber construction. Untreated piling cannot be counted upon for more than a year and ordinarily not for that length of time. Log rafts cannot be left in salt water for more than a few months or the logs will be ruined for lumber. This condition vastly increases the cost of all woodwork under water, diminishes its durability (even when carefully treated) as compared with the same work in fresh water, and is therefore a perpetual source of annoyance and cost. It will, of course, be entirely eliminated in the lakes.

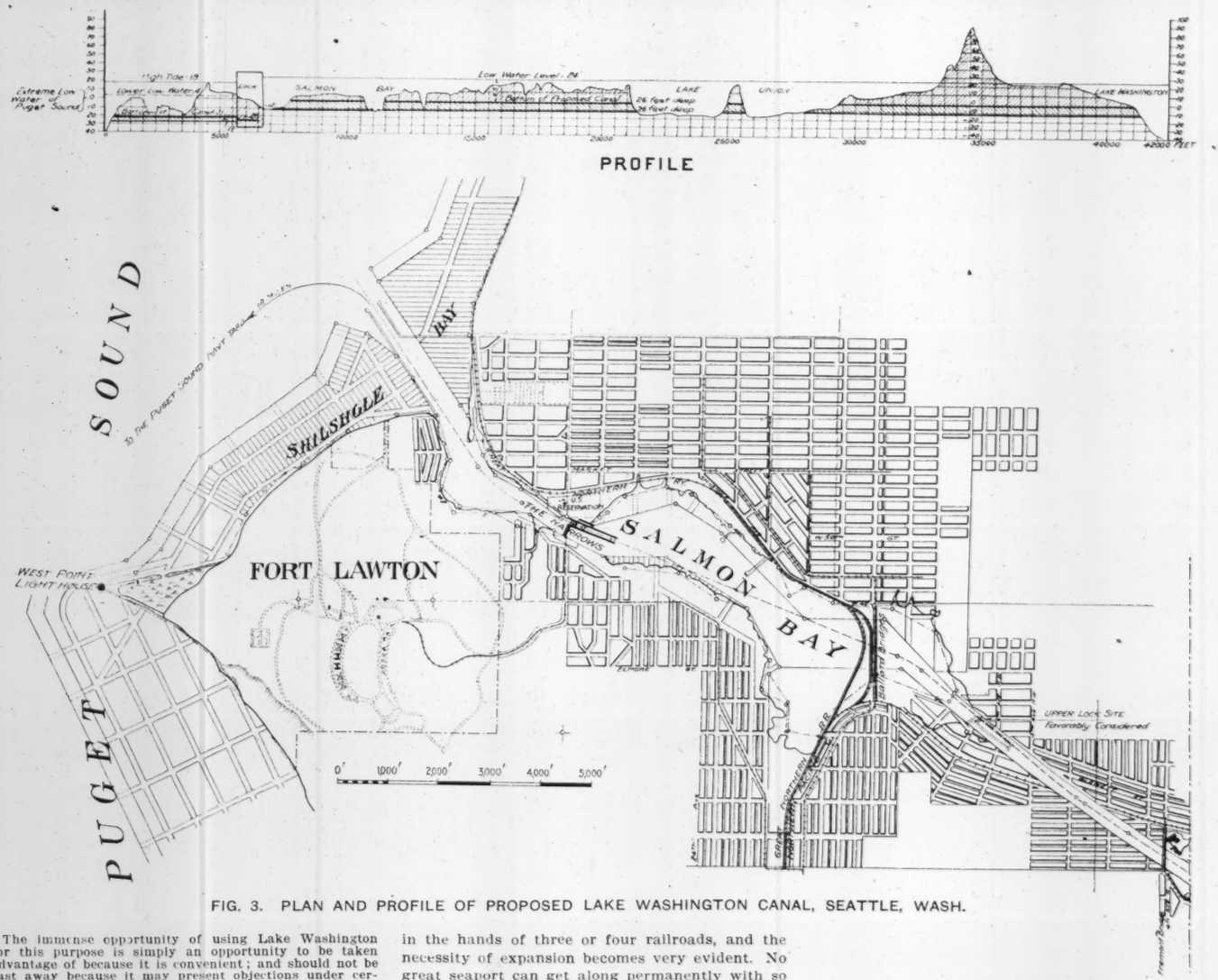


FIG. 3. PLAN AND PROFILE OF PROPOSED LAKE WASHINGTON CANAL, SEATTLE, WASH.

The immense opportunity of using Lake Washington for this purpose is simply an opportunity to be taken advantage of because it is convenient; and should not be cast away because it may present objections under certain circumstances.

To the average looker-on it certainly seems that if these bodies of fresh water are ever made safely accessible by a large canal the government is bound to make use of the opportunity thus presented to it. In fact, it would seem that on this ground alone the expense of building the canal would be justifiable as a national undertaking. Puget Sound is destined to become the second greatest, if not the greatest, naval rendezvous of the United States, and the advantage of free access to such bodies of water as these is not open to serious question.

From a commercial point of view, the canal means an enormous expansion of Seattle's harbor and the attainment of certain great advantages which the present harbor does not possess. Elliott Bay, is, to be sure, an exceptional natural salt-water harbor; but it is not without its serious defects. While its great depth makes it entirely free from the common drawback of shoal water, this depth is so excessive as to preclude the possibility of anchorage and to increase immensely the difficulty and cost of building piers in the hands of three or four railroads, and the necessity of expansion becomes very evident. No great seaport can get along permanently with so limited a frontage as this. For example, the water frontage of the Port of New York, on the Manhattan Island, Long Island and New Jersey shores, all of which is as available as the best four miles frontage in Elliott Bay, is upward of 50 miles.

The argument which applies to the water frontage of Elliott Bay applies equally to the availability of manufacturing sites. These cannot be had on tide water in the immediate vicinity of Seattle in sufficient extent to satisfy requirements. Lake Union and Lake Washington will meet this demand completely for an indefinite period in the future.

The mean tidal fluctuation in Elliott Bay is 10 ft. and the extreme 18 ft. All loading and unloading of cargoes, whether freight, or passenger; all mooring of vessels, whether rowboats or great ships; all wharf approaches and a multitude of other details have to take this condition into account. In the lakes this inconvenience is entirely absent.

Marine insect life in Puget Sound is extremely active and very destructive to all subaqueous

The deleterious effect of salt water upon the hulls of vessels had already been referred to. That the possibility of taking vessels into the lakes will prove of immense advantage in preventing the fouling of bottoms while lying in port and in removing growths which have already accumulated, is not open to serious question.

So far as the writer is aware, navigators have always regarded it as an advantage, whenever their vessels were lying in port for a time, to lay them up in fresh water. The nature of the service on this coast is such as to require many vessels to lie idle a considerable portion of every winter. Good anchorages in the Sound are scarce owing to its depth of water. The lakes will form ideal harbors of refuge at such times and be of immense benefit in the preservation of vessels.

The foregoing are some of the broad general grounds upon which this canal is justified as a public work. The benefits to the city of Seattle and surrounding territory are even more direct and vital. To a city of Seattle's topography (examine accompanying contour map, Fig. 2) the

problem of drayage must be in any case a costly one. Coal, sand, gravel, cement, lumber—all materials of extensive use in the daily life of the city and particularly in its growth and development—have to be hauled over hills and up and down steep grades to an extent hardly paralleled in any other city. In particular, the hauling heavy loads on such grades forces traffic over the streets with lighter grades and thus produces congestion which is often a source of great inconvenience and annoyance.

One has only to examine the map to see how great will be the relief when shipping can enter Lakes Union and Washington. Lake Union in

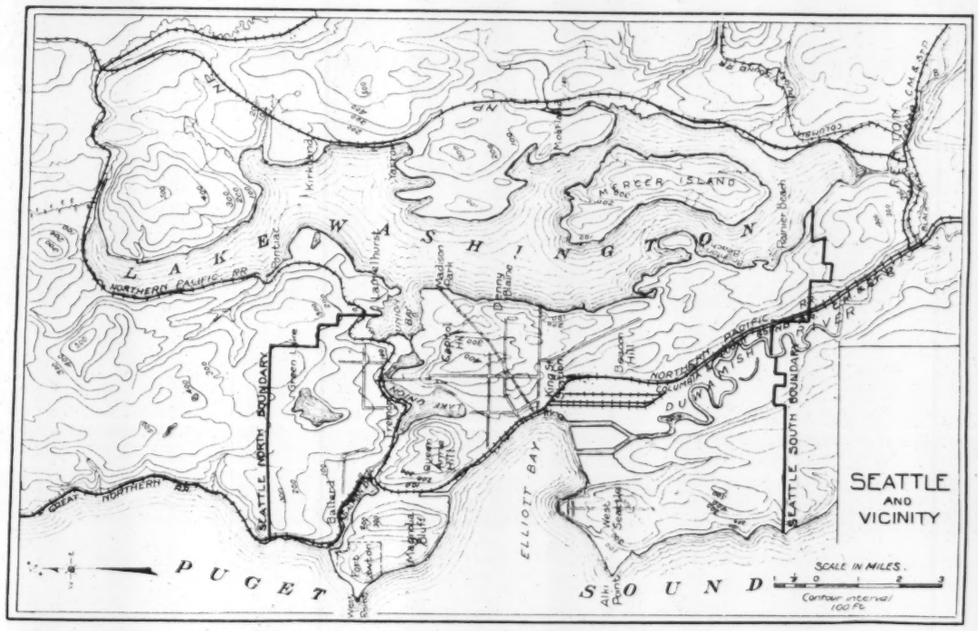
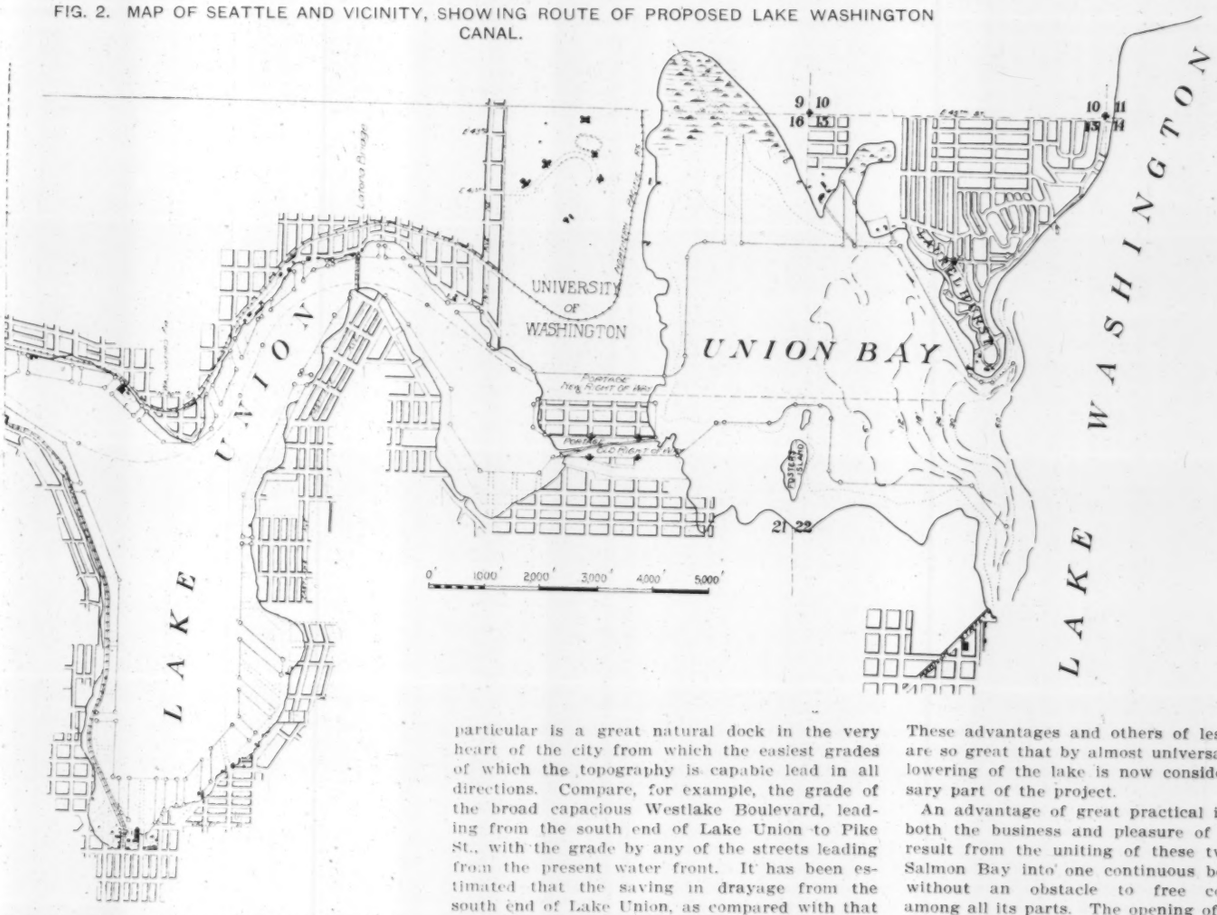


FIG. 2. MAP OF SEATTLE AND VICINITY, SHOWING ROUTE OF PROPOSED LAKE WASHINGTON CANAL.



streets leading from the present water front are nearly all of a very steep gradient, some of them as high as 20%, and therefore wholly impracticable for heavy hauling. The impossibility of particular is a great natural dock in the very heart of the city from which the easiest grades of which the topography is capable lead in all directions. Compare, for example, the grade of the broad capacious Westlake Boulevard, leading from the south end of Lake Union to Pike St., with the grade by any of the streets leading from the present water front. It has been estimated that the saving in drayage from the south end of Lake Union, as compared with that from the present water front, would amount to 25 to 50 cts. per ton. Owing to the large proportion of water-borne commerce this fact would control the drayage rates of the city. Alaska coal is almost certain to be an important factor in the future fuel supply of Seattle. Cement, sand, gravel and lumber, all come to a large extent by water. With Lakes Union and Washington open to shipping, railroad and drayage rates in and about the city would have to adjust themselves to the new situation, and the public would reap the benefit even upon traffic not coming or going by water.

Another important local advantage which will result from the canal is the improved drainage conditions around Lake Washington and the opening up of excellent factory sites now available. At the inlets of all the larger streams tributary to Lake Washington there are swamp areas caused by the deposits that have been brought down in past ages. At the south end of the lake where the outlet (Black River) is immediately joined by the considerable stream, Cedar River, the condition is very troublesome. High floods in Cedar River dam up Black River, force it back into the lake and frequently overflow the entire bottom lands between the Lake and Renton Junction in the Duwamish Valley (see Fig. 2). Lake Washington low-water level is about 8 ft. above the level of Lake Union. If Lake Washington were lowered by this difference and its outlet transferred to the canal, all of these swamp areas would be reclaimed. Cedar River would be carried directly into the lake, its overflow would be entirely controlled and the Duwamish River would be relieved of a considerable portion of its burden in time of flood.

These advantages and others of less importance are so great that by almost universal consent the lowering of the lake is now considered a necessary part of the project.

An advantage of great practical importance in both the business and pleasure of the city will result from the uniting of these two lakes and Salmon Bay into one continuous body of water without an obstacle to free communication among all its parts. The opening of a connection with the Sound will increase immensely the field of pleasure boating which already forms so large a part of the outdoor life of this community. Pleasure craft of all description will find in the

lakes their natural resort while laid up during the rainy season.

The lock (Fig. 5) is to consist of two parallel chambers, one large enough to take practically any craft, and the other quite small, designed to accommodate the "mosquito fleet." The dimensions of the large chamber are 825 × 80 ft., with 36 ft. draft, and of the little one 150 × 30 ft., with 16 ft. draft. Congress in the act ordering the survey and estimate, indicated its desire that the lock should be large enough to accommodate the largest vessels afloat, although it also authorized consideration of a smaller lock. Owing to the great probability that the Navy will make an extensive use of these lakes, it is certainly important to have the lock large enough to admit all of its vessels, and it was finally fixed at the above dimensions, which will admit anything except such monster craft as the "Lusitania" and the larger Dreadnoughts. The length of the lock (greater than required for vessels) was adopted for the purpose of passing the largest log rafts on the Sound without breaking. The chamber has a middle gate so that its length can be shortened in operation if desired, the whole combination giving practically four lock chambers of 150, 350, 475, and 825 ft., respectively. The lock will be one of the notable structures of its kind in the world. Considering collectively the three dimensions, length, breadth and depth, it will exceed in size any lock heretofore constructed; it may be equaled by one contemplated European project and exceeded only by the Panama locks. At mean sea level there will be 36 ft. of water on the lower sill. The lift of the lock will be 22 ft. from mean lower low water to high water in the canal, 10 ft. from mean higher high water and 15 ft. from mean sea level.

A feature of the plan is found in the power plant provided at the lock site. The entire flow from a drainage area of nearly 600 sq. mi. will pass the lock site and drop to sea level with a mean head of 15 ft. The great reservoir formed by the lakes, and also by Sammanish and Cedar Lakes on the streams of the same name, makes possible a very complete regulation of the discharge. It is probable that 1,000 effective horse power can be developed, except during 75 days (in the dry summer months), and this will always find a ready market, located as it is in the heart of a great city. The revenue from it should go a long way toward paying the cost of operating the canal.

Owing to the practical impossibility of enlarging a lock when once built, except by complete reconstruction, the plans call for ultimate dimensions to begin with, though doubtless a smaller structure would answer for some time to come. The canal section, on the other hand, can be enlarged at any time without a suspension of traffic, and it is therefore proposed to excavate only to a depth of 25 ft., and bottom width of 75 ft., at present, leaving the final enlargement to 36 ft. draft and 100 ft. bottom width until the needs of commerce require it.

A source of much controversy and the foundation of some recent opposition to the canal arises from the location of the lock. As proposed in the official project it is at the foot of Salmon Bay and at the site selected by the Government in 1891. Flowage rights were purchased and paid for around the shores of the bay to admit raising its surface to the level of Lake Union; but the Government has been so slow in commencing the actual construction that during the elapsed ten or twelve years there has grown up in the minds of some influential property owners and manufacturers the belief that they have acquired vested rights. These interests and some others actuated by sympathy or personal profit,

FIG. 4. LAKE UNION AND

(Lake Union in foreground, Queen Anne Hill in center background. Olympic Mts. dimly in distance. Puget Sound in background to right and left of Queen Anne of picture to right. Elliott Bay lies mainly concealed by low ridge to left

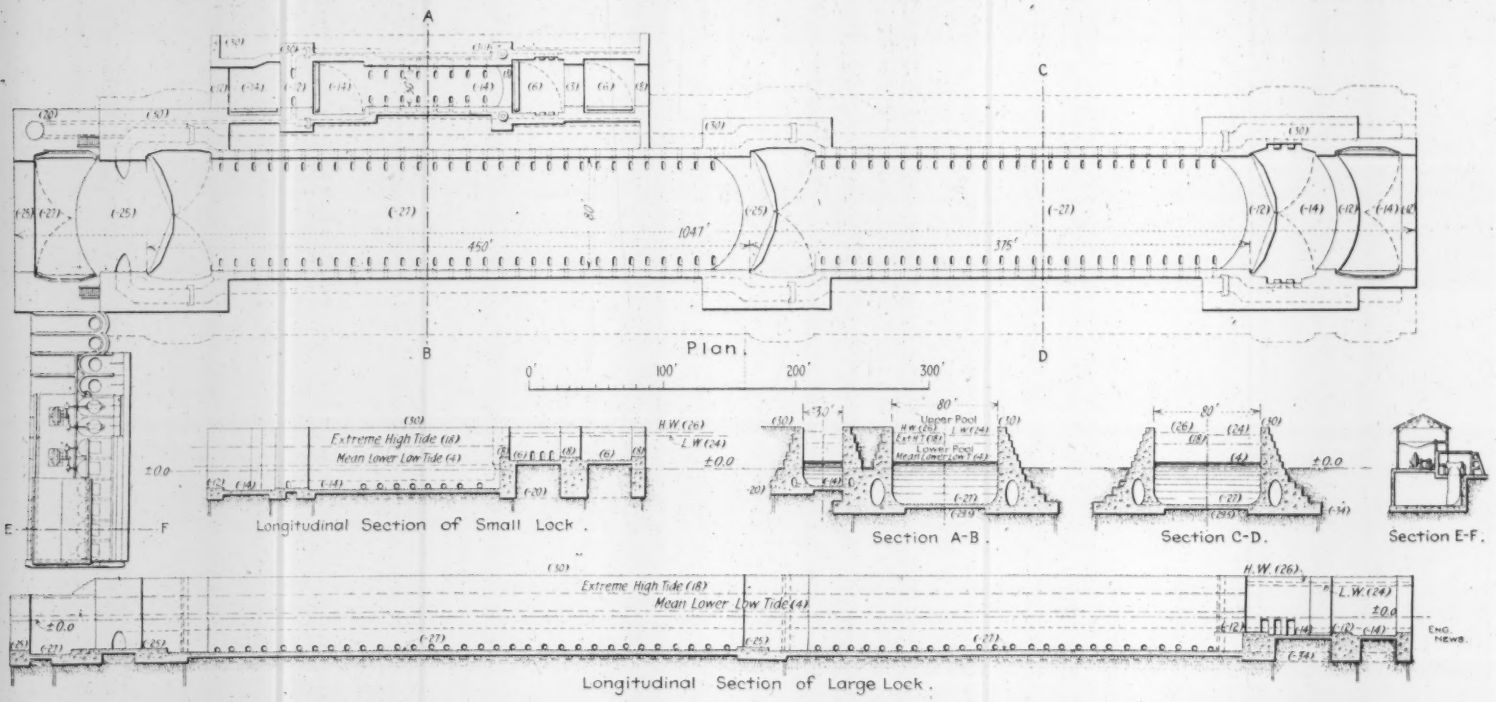


FIG. 5. PLAN AND SECTIONS OF LARGE AND SMALL LOCK IN PROPOSED LAKE WASHINGTON, CANAL, SEATTLE, WASH.

abetted by those who wish to avoid friction, seek to force the location of the lock to the head of Salmon Bay. While the mill owners insist now upon being left undisturbed there is not the smallest evidence of willingness to refund the money received for the flowage rights. This fact puts them in an attitude it is hard to defend from any point of view. It is not an uncommon occurrence for individual interests to be magnified and public interests minimized, as is being done in the case at hand. It is an instance where successful business men, and leading citizens, too, have become, through intense application, so absorbed in their own affairs that their vision is narrowed and they fail to take a broad or just view of the situation. The removal eastward of the lock would prove unfortunate to the interests of the public by militating against the best development of Salmon Bay, by retarding the full usefulness of the canal and by immensely increasing the ultimate cost of the waterway.

Referring to the second purpose of this paper, mentioned in its first paragraph, the question naturally arises: why has so simple a project as this, and one that is fraught with so many obvious advantages of a public and local character, been so long in being carried out? The answer is to be found in the power of private interests, which imagine that they may be adversely affected, and which in some instances will be adversely affected, to resist successfully the general demand of the public. The vast majority of the local community believe in the work and want it to be carried out. In 1906 the question came before the voters of King County in the form of a proposition to raise money in aid of the canal and carried by an almost unanimous vote.

So great are the local advantages which will result from the canal project that it is considered that a portion of the expense of the work should be borne by the local community, and the local community has always accepted this situation. Several years ago it acquired at a cost of about $250,000 the complete right-of-way and delivered it to the Government. In the Government engineer's report recently submitted to Congress it is recommended that the further local contributions consist of digging the channel and that the Government build the lock and do certain other portions of the work. The state has made an appropriation of $250,000 and local interests are now engaged in raising $1,000,000 more through an assessment district to cover the estimated cost of excavation. This, with the cost of the bridges which the city will have to build, and the rights of way already paid for, will just about equal the cost to be borne by the Government, about $2,500,000, thus making the Government and the local community practically equal co-partners in a five-million dollar enterprise. The future operation of the canal will be borne by the Government and the canal will be toll free.

The above scheme for providing funds for building the canal has been sanctioned by the Government officers and received the approval of members of Congress interested in meritorious river and harbor work. In fact, the great strength of the movement rests in its advocacy by legislators from eastern and middle western states who recognize the national import of the improvement and its influence in developing the Northwest and facilitating commerce on the Pacific Coast.

The project is in every sense a high-grade public enterprise. To the Navy of the United States it will be of undoubted value. As a means of improving harbor facilities and leading to the development of shipping and industries it is as meritorious as any project of harbor improvement in the United States. Locally, it will save every five years, and probably oftener, to the city of Seattle and its residents in the cost of drayage alone the total sum asked of them in aid of the project. It will accomplish swamp reclamation and flood control which would cost immense sums to accomplish in any other way. It will be of value in the sewerage problems of the city, and it will greatly enhance the value of outdoor enjoyment of the people by practically doubling the field of pleasure boating. And yet this important measure has been held up by influences which the most charitable judgment cannot pronounce other than selfish influences that proceed solely from the assumed injurious effect upon particular private interests. The only exception is the honest fear which some people entertain that the changes proposed will injuriously affect the scenic value of Lake Washington. As this is a fear arising from high motives of public good a special word will be devoted to it here.

It is not believed that the building of the canal will produce the effects feared in the smallest degree. Even at the first, the changes will be discernible only upon close inspection, and not at all after a lapse of one or two years. The drainage of swamp areas will improve rather than detract from the natural beauty of the shore line. Along the bluff shores of the residence districts the narrow belt that will be bared by lowering the lake will be quickly converted into turf and shrubbery and will produce a substitute as good as, and perhaps better than, the present, which is bound to be greatly modified by occupation and settlement. Factories will not seek the residence districts because of unfavorable topography. The sites which they would naturally seek will otherwise be occupied by inferior establishments, just as they have been around Lake Union, and the situation will be improved rather than aggravated by the substitution of important factories. Finally, those who feel that the presence of ships on the lakes will detract from their interests have an ever present refutation of their fears in the Queen Anne Hill district of their own city. To the occupants of the hundreds of elevated residences overlooking Elliott Bay there is no greater attraction in the landscape than the maritime activity that prevails on the bay. No one ever hears it remarked that the shipping is an objectionable feature there. In short, all these fears are ill-founded. Lake Washington is too large a body of water to be seriously modified in appearance by anything man can do, and it will remain after the canal is built, what it is now, one of the chief beauties of a very beautiful city.

PORTION OF SEATTLE, WASH.
Hill. Canal route between Sound and Lake lies in low depression to right of Queen Anne. Route to Lake Washington is by way of arm of Lake Union passing out of Queen Anne and is one mile distant from Lake Union.)

A Rubber Stamp for Inking-In Field Rivets on Structural Drawings.

By A. L. CAMPBELL.*

In standard drafting-room practice, those rivets in a structure which are to be driven in the field after assembling are represented by circles, of the approximate diameter to scale of the rivets, inked in solid.

Where speed is more essential than fine drafting, field riveting may put on a tracing as follows: Procure a penny pencil with a cone-shaped

Rubber Stamp for Field Rivets. Made of a Lead-Pencil Eraser.

rubber eraser inserted directly into the wood at one end. Cut this rubber off square to the size desired, as shown in the accompanying sketch. Then make an ink pad by saturating a piece of blotting paper with drawing ink, and apply the rubber alternately to the ink pad and tracing. When the rivet centers are already located a fair job can be done very quickly with a little practice. Three of these pencils cut to make three sizes of rivet holes will fill all ordinary requirements and will last a long time.

SPECIFICATIONS FOR FIRE HYDRANTS of the post or above-ground type have been prepared by a committee of the New England Water-Works Association, and will be submitted for discussion at a meeting of the association to be held on Jan. 12. Printed copies of the proposed specifications have been sent to members of the association, and can doubtless be obtained by others interested on application to the association at Tremont Temple, Boston.

*Solvay Process Co., Detroit, Mich.

A New Process in Iron Metallurgy; Slow-Rusting Iron Produced in the Open-Hearth Furnace.

A remarkable iron has been produced and sold for a short time past by the American Rolling Mill Co., of Middletown, O. It is a product of the open-hearth steel furnace, but in chemical composition and in mechanical properties it is so closely similar to wrought-iron that the term "steel" can no longer be applied to it properly. The makers have given it the trade name "American Ingot Iron," and for the purposes of this article we may suitably speak of it as ingot iron, though it is possible that a more characteristic term for a product of that nature may be found.

This ingot iron differs from steel in having its carbon, manganese and silicon contents reduced to small fractions of the amounts found in ordinary soft steel. A typical analysis is as follows:

C*	0.02
Mn	0.0.
P	Tr.
S	0.02
Si	Tr.
O†	0.03

*By combustion.
†By ignition of 25-gram sample in stream of hydrogen and absorption of water in phosphoric anhydride (method of Ledebur).

The metal is to all appearances identical with dead-soft steel or a good wrought-iron, though its fracture lacks the ropy fiber seen in the latter. Its strength is closely the same as that of wrought-iron. Several tests indicate an ultimate tensile strength of from 47,000 to 49,000 lbs. per sq. in. Its ductility is remarkably high, and the metal is rolled into sheets and the sheets used for drawing in the drawing press with good results.

The prime quality of the ingot iron, however, is its resistance to corrosion. This was the object for which it was originally produced as an experiment, and when trials against steel in sulphuric-acid solutions showed the hope of high corrosion resistance to be borne out, it made the metal at once the most interesting appearance in iron metallurgy of the present time. On this account we desire to give here a statement of the process of manufacture, and its development from ordinary basic steel production. The data for this statement were obtained on a recent visit to the plant of the company, at which time all information was freely afforded us.

It should be remarked at the outset that the work of development in connection with the ingot iron is still being continued. This leaves the possibility of future improvements, for example in strength. But the present stage is a definite one, for two reasons: In the first place, the method of working has been so thoroughly tried out and perfected that excellent workability and a high degree of uniformity of composition are obtained in the regular operation of the furnaces producing the iron. In the second place, the process has been individualized by two patents recently issued on process and product. As set forth by these patents, the process can be concretely defined, and produces a definite material which to the best of our knowledge had not previously been made. Indeed, neither the desirability of having such a material nor the possibility of producing it had been recognized. We shall have occasion a little farther on to quote a description of the process from the patent claims.

Evolution of the Process.

The American Rolling Mill Co. began the manufacture of plain and galvanized steel sheet in 1901. The works were planned for basic open-hearth steel, although at that time very little sheet was rolled by others from basic open-hearth metal, and in particular it was doubted, and in some quarters denied, that sheets suitable for galvanizing could be made from such metal. The plant therefore began with an innovation, and it appears that this bears more or less directly on the subsequent development of the ingot iron.

AIMING AT LOW IMPURITIES.—The use of basic open-hearth sheets for galvanizing being quite new, special consideration was given to the question of what grade of metal to produce. It happened that the superintendent of the furnace, Mr. R. B. Carnahan, Jr., had previously been employed both in open-hearth work and in puddling-furnace work, the latter in connection with the manufacture of high-grade charcoal iron sheet. He held the idea, therefore, that the way to success in the new plant was to aim at the composition of high-grade wrought-iron as an ideal; that is to say, to produce a low-carbon metal as soft and as pure as was feasible.

The earliest product was an ordinary soft steel, running about 0.20% C, 0.40 Mn. This metal was successful both in the mills and in the galvanizing department, but the ideal of purer metal was not forgotten. Attention was directed mainly to the carbon as the impurity to be reduced, and after some time of working the carbon was brought down to 0.10%, giving a dead-soft metal. This steel was rather high in phosphorus, averaging perhaps 0.075 P, which prevented trouble from sheets sticking together in the sheet mill. The soft sheets rolled from this metal gave the company a special reputation, to the extent at least that its product commanded a premium in the market for galvanizing purposes.

A change in the work of the plant was initiated in 1904, by the requirements of a newly developing business, namely, the manufacture of corrugated steel culverts. The company had previously produced only thin sheets ranging from 20 to 28 gage. Culverts required heavier gages, and when the selling department saw the possible market it called for sheets of 14 and even 12 gage. In connection with this, the question of making a special metal for the culvert sheets was given consideration. It was of prime importance to produce a metal of highest possible rust resistance. There was then, and is to-day, a very widespread conviction that puddled wrought-iron rusts more slowly than mild steel. The new demand was thus in harmony with the previous ideals of the plant managers to approximate a wrought-iron composition, and an added impetus was given to the experimental reduction of impurities in the furnace.

The result was that thereafter the furnaces were driven to a higher reduction of impurities, and the ferromanganese addition for clearing and recarburizing was cut down to what was thought a minimum (consistent with producing workable metal). Steel running as low as 0.08 to 0.10% C, 0.16 to 0.20% Mn (occasionally as low as 0.10%), and 0.03 P, was thus obtained. The simultaneous reduction of manganese and phosphorus, it seemed, prevented trouble from sticking of sheets during rolling. Cracking of sheets in the rolls was counteracted by the addition of materials such as ferrosilicon in the bath, to burn out the oxygen, and additions of aluminum in the ladle to remove gases.

THE IDEA OF ZERO MANGANESE.—The lowering of manganese content had not up to this time been done as a primary matter, but was merely part of the general reduction of impurities. Then came a suggestion that manganese in itself was the most important undesirable element. This was contained in a Department of Agriculture publication issued about 1904, a monograph by Mr. A. S. Cushman, which dwelt forcibly on the rapid rusting of wire fences and, tracing the short life of modern steel wire (compared with the iron wire of an earlier period) to its manganese, urged that steelmakers should attempt to produce a metal low in manganese. The operating officials of the company were much impressed with this advice, and though they did not believe that further reduction of manganese was feasible without destroying the rolling quality, they set out to make trials. To produce roofing sheets and culvert material of increased resistance to rusting seemed so important that they could not afford to neglect the chance of a trial.

Following out this purpose, successive reductions of manganese by longer oxidation in the furnace and less recarburization brought them to a stage where metal with the extremely low amount of 0.07 Mn was obtained. No evil results were found, none at least that could not be corrected. To go lower, however, seemed to mean leaving out the ferromanganese recarburizer entirely. After many doubts, this step was finally resolved upon, and the first melt was tapped out without recarburizer, giving a metal of practically zero manganese. The product was not ideal in the rolls, but the prior experience in bringing out the steels of 0.10% to 0.15% manganese was helpful enough to make the result promising for fuller success. From that time on, all melts were run without using ferromanganese.

PERFECTING THE PROCESS.—It was a first consideration of the works manager to produce serviceable metal at all times. He could not afford to interrupt the operation of the mills by producing a heat of worthless metal that would have to be remelted. With 50-ton furnaces, and only two of these, both being worked continuously to furnish ingots to the bar mill, there was little margin for doubtful experimenting. The development therefore had to progress in small stages, cautiously, going ahead from each step only so far as seemed safely practicable. This will serve to account for the fact that more than two years was consumed in eliminating the last 15 points of manganese.

After the first no-manganese metal had been brought out, there followed a long time of struggling with defective metal. The percentage of sheets spoiled in rolling or galvanizing was ruinously high at first, and improvement of quality was absolutely necessary. But it was seen that the process had promise, inasmuch as some of the metal was workable, and the troubles were of the same kind as those encountered and overcome previously, in the reduction of manganese. Moreover, the new product soon proved to be valuable, since its corrosion-resisting powers met all expectations.

Since that time these troubles have been overcome, as a result of accumulated experience in working the process. These difficulties were mainly two: cracking at the edges in rolling the bars into sheets, and poor galvanizing quality due to blistering. The way in which these were overcome must be judged from the fuller description of the process below. Briefly stated, it was learned by experience that the old idea of the need of manganese for giving good working quality in the hot condition is erroneous, and that the use of materials for purifying the molten metal of oxides and clearing it of entrained gases accomplishes the same results.

A peculiar observation must be noted here, namely, that, as the company found, it was not sufficient to produce a metal of good rolling qualities; for even though sheets might roll well, they might be failures in the galvanizing process. Since a large part of the product of the company's mill was used up in its own galvanizing department, the trouble of spotted or non-adherent zinc coating soon developed, and it gave fully as much trouble as the matter of lack of ductility in rolling. It is probable that, had the company been in the business of rolling black sheets only, the development of the process would have been slower.

The Process of Manufacture.

The preceding sketch of development would hardly suggest that the process by which the ingot iron is made is essentially very simple. This is the fact, however. The same basic-lined furnace is used as in regular open-hearth practice, and practically the same charge is made, but the boiling is continued to an excessive degree, so as to oxidize the impurities more thoroughly than in ordinary steel practice. Naturally the process in its final stages goes on at a much higher temperature, on account of the higher melting-point of the purer material. The usual ferromanganese recarburizer is omitted, and instead an addition of ferrosilicon or an equivalent substance is made to purify the bath of its entrained oxides. Finally, aluminum is added in the ladle, to carry out the gases dissolved in the fluid metal.

We find a more exact definition of the process

in the claim of Patent 940,784, issued Nov. 23, 1909, to Robert B. Carnahan, Jr., the company's superintendent, as follows:

> Charging a high-powered basic open-hearth furnace with iron; employing therewith eliminating agents for carbon and manganese and sulphur and phosphorus; refining the charge in molten condition at such high and increasing heat and for such prolonged period that analysis by bath test shows that the sulphur, phosphorus, carbon, manganese and silicon are not over 0.14% when taken in the aggregate; and pouring the hot purified metal into molds, and treating the metal while in molten state with a deoxidizing and degasifying agent or agents to such extent that the product will contain not over 0.05% of oxygen and will contain at least 99.80% of iron, substantially as set forth.

A more specific description of the manufacture is best given by stating details of the process as worked and as recorded in the books of the company.

RAW MATERIAL.—The furnaces now in use work entirely on cold pig-iron, all of which must be bought from outside parties. The quality desired is what is known as a high-grade northern pig, though Alabama iron has been used to some extent. A grade slightly above the limiting phosphorus content for Bessemer working is used, but low silicon and low sulphur are special desiderata. Two typical analyses of the pig-iron received at the works are:

Si	0.89	0.77
P	0.19	0.152
S	0.018	0.022
Mn	0.78	0.70

A fairly large proportion of scrap is charged. Some of this is regular open-hearth scrap, chiefly the waste from the mill itself; the larger part, however, is steel turnings, representing, of course, also, a low-carbon steel of good grade.

Some ore is charged, both in lump and in fine form. As this is not so conveniently obtained, however, mill scale from the bar mill has been tried, and it has been found fully as effective as the ore; so that recently, whenever sufficient mill scale is on hand, this has been used largely in place of ore.

To produce a slag, limestone and fluorspar are used. The high temperature maintained toward the end of the process, when the metal is so much purified as to have a high melting-point, favors the return of phosphorus from the slag to the metal, and therefore requires comparatively large quantities of the fluorspar. A rather deep slag is carried.

The essentials of the melting process are the maintenance of an active basic slag, continuance of the boiling process until all foreign elements in the iron are reduced practically to zero, keeping up a sufficiently high heat for complete fusion and active boiling, and finally the treatment for removing oxygen and other gases by addition of purifiers.

In the removal of the impurities, ordinary oxidation and absorption by the basic slag plays the chief part, as usual. It is found advantageous, however, to add ore for helping with the oxidation. When the pig runs high in carbon, especially, a charge of lump ore is made at the start to burn out the major part of the carbon. Under other circumstances only fine ore is used, being added in the later stages of the boiling.

The removal of oxygen (presumably contained in the metal as oxides) is most important. When this part of the process is imperfect, the metal is red-short and cracks in rolling. In ordinary steel-making, spiegeleisen or ferromanganese have the chief duty of burning out the oxygen, reducing it from the metal both through their carbon and their manganese. (It is recorded that the manganese contained in recarburizing additions appears only to a small extent in the metal tapped, while a large part promptly goes into the slag as oxide.)

In the present ingot iron process, the removal of the oxygen can be accomplished by various substances, it has been found. Pig-iron in itself is a satisfactory means, its carbon and silicon doubtless playing prominent parts in the reducing action. Ferrosilicon is a more active agent, and at the American Rolling Mill Co.'s plant this is largely used. Vanadium, titanium and other purifiers have been tried, without such results, however, as to warrant their permanent use in place of ferrosilicon.

The removal of other gases, except as these in part may also be removed by the ferrosilicon in the bath, is done by adding granular aluminum in the ladle. Aluminum has for some time past been used very widely in steel-furnace practice for this and related purposes, and doubtless its function in the manufacture of ingot iron is in no way different from that in regular open-hearth practice. In a large number of heats whose records were inspected the aluminum addition ran usually well below 0.1% of the weight of the iron charged, though in special cases it runs as high as 2½ lbs. per ton, equivalent to 0.125%. These additions are not made on a calculated basis but are gaged by experience. The aluminum is thrown into the ladle as the charge is tapped, and when in the first ingots poured the high shrinkage indicates imperfect elimination of gases a further addition is made in the pouring gate.

The temperature just before tapping the ingot iron is required to be 3,000° to 3,100° F. The furnace is observed with a Fery radiation pyrometer at the end of each melt, before tapping off is begun, to make sure that the iron is not tapped too cold.

TYPICAL MELT RECORDS.—A better survey of the working of the process may be had from some records of representative melts, selected at random from the books covering the period of recent weeks. Five such records are given below.

RECORD OF FIVE HEATS IN 50-TON FURNACE NO. 2, AMERICAN ROLLING MILL CO., MIDDLETOWN, OHIO.

	1 Nov. 2	2 Nov. 14	3 Nov. 20	4 Nov. 29	5 Dec. 2
Charge:					
Pig	50,000	54,300	58,000	53,000	30,000
O. H. scrap and turnings	70,100	45,700	62,000	67,300	49,900
Total iron	120,100	100,000	120,000	120,300	79,900
Ore	1,000	1,000	1,000	.....	1,000
Fluorspar	1,000	1,000	2,200	.....	1,000
Limestone	13,500	10,000	10,000	13,300	8,000
Mill-scale	3,000	3,000	.....	.....	.....
Purifiers:					
Ferrosilicon	200†	100‡	{175† {175‡	300‡	250‡
Aluminum	75	62	75	90	60
Time of melt	10h40m	10h15m	9h30m	10h40m	.....
Metal:					
Weight of ingots	96,000	80,000	98,000	96,000	64,000
Analysis:					
Sulphur	0.024	0.024	0.028	0.022	0.027
Phosphorus	0.005	0.007	0.002	0.004	0.003
Carbon	0.01	0.03	0.02	0.01	0.01
Manganese*	0.03	0.02	0.01	0.01	0.03
Si	(Average, 0.005)				
O	(Average, by hydrogen method, 0.03)				

*In many other analyses the manganese is reported only as "Trace." The amounts shown in the above five heats are high.
†90% silicon.
‡50% silicon.

It will be seen at once that the time required for running a heat is very considerably in excess of that needed in ordinary open-hearth steel practice. We have here an average period of ten hours from beginning of charging to tapping, an increase of several hours over steel furnace practice. This is altogether traceable to the thorough-going elimination of impurities. Evidently this work calls for long-continued boiling, in conjunction with suitable regulation of the slag. This long heating in an oxidizing flame inevitably produces what would ordinarily be regarded as an overheated or burned condition of the metal. It is indeed a characteristic of the process, as regarded by the responsible managers of the American Rolling Mill Co., that the bath is intentionally superoxidized. As expressed by Mr. Carnahan, the chief figure in the development of the process: "*We do with deliberation what the steelmaker with deliberation avoids, namely, burn the bath. That is the first characteristic of our process.*"

The long duration of the melt necessarily increases the cost of the product materially over that of ordinary steel. Increased expenditure for fuel and, more important, increased plant and labor charges—the average output being admittedly brought down to two-thirds the usual figures—must appear in higher cost. In considering this phase of the matter, however, it is well to bear in mind the charge frequently brought against steel-furnace practice—that modern working is far too rapid for best quality of metal, and that slower working, both in the furnace and in pouring, and in rolling, could be relied on to produce metal of higher quality. It is an open question, therefore, whether a part of the increased time (and hence cost) of working in this ingot-iron process should not be deducted, in view of the fact that in this process no consideration of hurrying the output is allowed to interfere with the operation for a product of best quality.

FURNACES AND INGOTS.—The two present furnaces, each of 50 tons capacity (a third is building and is nearly completed), are designed specially for high power, i. e., to enable high temperatures to be attained. The high temperature is a requisite of the process, but no special construction seems to be needed for the purpose, as most modern large-capacity open-hearth furnaces can give the desired temperature.

Natural gas is used as fuel. This is of advantage both because of the high thermal value of the gas and because of its relative purity. But producer gas is able to give equal results, as was found at Middletown in an early stage of the development, before natural gas had been obtained.

The present plant pours all its metal in small ingots, about 1,200 lbs. each. These are approximately 8 × 12 ins. by 4 ft. They are bottom-poured in groups of 20 from one gate. The size of ingot is limited by the capacity of the present bar mill, which produces sheet-bars direct from the ingot. It is planned in the new works (of which a few words will be said later) to employ large top-poured ingots, 20 × 24 ins. by 6 ft.

No trouble from blowholes has appeared as yet. Nevertheless the elimination of gases goes on actively even in the molds, especially when the aluminum addition was rather low. In the latter cases a deep open shrinkage pipe forms in the head. It is possible to secure perfectly flat-topped ingots by using enough aluminum, but this is considered undesirable, as the ingots do not roll as well; a pipe 1 to 2½ ins. deep is preferred. The depth of pipe is currently used as index of the sufficiency of aluminum addition, and in some cases, when the first ingots poured show too deep a pipe, more aluminum is thrown into the ladle or the pouring gate for its effect on the rest of the metal.

FURNACE WEAR.—The high temperature carried to melt the low-carbon iron is quite destructive to linings, brickwork, etc., so that these parts require more frequent renewal than in steel practice. Ladle-linings often last only three melts. Ingot molds crack much sooner than with steel. The furnace roof itself is destroyed two or three times as fast (measured in number of heats, not in total time) as when working on steel. The whole furnace must be repaired and the roof completely renewed about every six weeks. These items of course add to the increased cost already mentioned.

Results of the Process.

UNIFORMITY.—The important question of whether this superoxidation process—for the Carnahan process is essentially that—is able to give uniform and hence trustworthy results finds answer in the series of analyses tabulated above. They show total impurities (except oxygen) varying between 0.05% and 0.08%, a fairly close range. To amplify this table it may be said that the same range of impurities has been maintained for many months. The results in the rolling, galvanizing and corrugating departments of the works also indicate that satisfactory working quality has been maintained quite uniformly. Thus, with watchful, intelligent control the process is in no way erratic or variable, and the highly pure metal shown by the above analyses can be regularly produced.

STRENGTH OF METAL.—Tests of mechanical strength of ingot iron are as yet too few to quote. The best evidence obtainable points to strength and ductility identical with those of wrought-iron, i. e., an ultimate strength of 47,000 to 50,000 lbs. per sq. in. and high ductility. Under ordinary circumstances this would lead

us to expect the yield-point to lie at 25,000 to 30,000 lbs. per sq. in., corresponding to the usual elastic ratio of about 0.6. But four 1 × 5/16-in. bars tested at Columbia University by Prof. Woolson at our request gave a yield-point of 39,000 to 42,000 lbs. per sq. in., with ultimate at 48-49,000. This unusual ratio might be taken to indicate hard metal, but the ductility as shown by reduction of area (80%) was unusually high, and the elongation was good.

In substance, as regards strength the metal is to be considered the equivalent of wrought-iron.

RESISTANCE TO CORROSION.—Naturally the special resistance of the ingot iron to rusting makes this the most interesting property to inquire about. But, unfortunately, the subject of corrosion and corrosion-testing is so intricate that specific figures are worthless unless they be averages of very many tests made under many different conditions. In proof of this we need only point to the diametrical opposition of views on the relative rusting resistance of steel and wrought-iron.

The difficulty of conducting a fair comparative test of rusting under atmospheric exposure is so great, and such tests are so slow, that we need not wonder at the lack of test-results in the case of ingot iron. The only available information is that derived from tests in sulphuric-acid solutions, and such results can only be inferential in respect to atmospheric rusting.

Quite a number of sulphuric-acid tests have been made, in which ingot iron and mild steel were immersed in equal solutions for equal lengths of time. Various figures of the results have been published, showing generally a resistance forty to sixty times as great in the case of ingot iron as in the case of steel; some, in which samples of wrought-iron were also included, showed the ingot iron to exceed wrought-iron twenty to forty times. We need not lay special stress on the precise figures recorded, in view of the inherent uncertainties of such tests. The ratios mentioned are so large that we can safely draw the conclusion that ingot iron in sulphuric-acid tests resists many times as well as either steel or wrought-iron.

More important still, the large ratios of resistance make it permissible to transfer the comparison to *atmospheric rusting*, after making a large reduction as a safety-margin. We conclude in this way that the ingot iron will resist atmospheric corrosion at least several times as well as mild steel, probably many times better; and markedly better than good grades of wrought-iron.

The matter is so weighty, however, that it should not permanently rest on a basis of estimated comparison, but should be confirmed and rendered definite by careful and broadly-planned tests conducted by competent independent authorities.

OTHER QUALITIES.—An interesting observation made in the works of the company itself has some connection with the subject of corrosion. The feed-rolls in the galvanizing tanks where the sheets are coated with spelter are driven by gears which are immersed in the molten metal. The wear of these gears is quite rapid, doubtless because of chemical actions, for the face pressure on the teeth is low. Formerly cut steel gears were used. As the company has for some time made castings for various works purposes of steel and ingot iron, it was natural to think of trying ingot iron for these gears. The material stood up considerably better than steel, the life of the gears being from two to six times as long. The gears are now regularly made of ingot iron. For this purpose an ingot is forged round, and cut into disks by a band-saw; a hole is bored in each disk for the shaft, and the teeth are cut on a gear-cutter. Local circumstances make these gears rather costly, but as their longer life gives fewer interruptions to the working of the galvanizing pots their cost is of little moment.

The ingot iron has been found to possess an excellent degree of weldability, substantially equal to that of high-grade wrought-iron. This quality has led to the use of ingot-iron sheet in cases where it was necessary to weld up seams, as in forming tanks.

In taking a good coating in the galvanizing process the ingot-iron sheets have proved superior to charcoal iron, according to the statement of the company based on experience in its own galvanizing department. After the early troubles with spotting and non-adherent patches, which was overcome by more thorough reduction of oxides and gases in the furnace process, the results in galvanizing have been uniformly excellent.

Ease of enameling has also been found to be a special feature. Makers of enameled sheet ware (baked enamel) have preferred it for this reason to steel and iron.

Commercial State of the Process.

Up to the present, ingot iron has been produced for the market only in the form of sheets. The furnace capacity of the Middletown plant is only large enough to produce material for the requirements of sheets. However, it is clear that other manufactured forms offer as valuable a field for a slow-rusting iron.

The manufacture of wire has been in contemplation, in response to numerous demands for ingot-iron wire. The works having no equipment for wire-drawing, or even for producing wire rod, it has not yet been possible to produce wire even for experiment. We understand, however, that it is planned to have wire drawn from ingot iron in the near future, primarily for test purposes, with a view to developing a wire business when the company's new plant is in operation.

Rolled structural plate and shapes appear to need a slow-rusting metal just as urgently. The enormous extent of exposed metal in bridges, tanks, smoke-stacks and dozens of other structures which stand in the open makes the reduction of loss by rusting a consideration of first importance, and therefore creates a vast field for a metal which will give several times the life of ordinary steel. The new works to be built next year contain provision for the manufacture of plates, and it is to be hoped that shapes will also be rolled.

A brief statement of the equipment of the works where ingot iron is produced may be of interest.

The Manufacturing Plant.

OPEN-HEARTH DEPARTMENT.—Two 50-ton-basic open-hearth furnaces in operation, one additional under construction (nearly completed). They are fired with natural gas but are equipped also for producer gas. Electric chargers are used. The runway over the pit carries a 75-ton and two 15-ton electric traveling cranes.

ROLLING MILL.—The bar mill comprises a three-high roughing mill and a three-high finishing mill, capable of working 8-in. ingots into sheet-bars. These mills are now being fitted with automatic tables. The ingots are heated in two large regenerative furnaces, being charged and removed by a revolving electric charger.

The bars are worked up in four sheet-mills supplied by eight heating furnaces. A large annealing room with six double-annealing furnaces handles the sheets produced. More than half the output of bars, however, is sent to Zanesville, O., where the company (since 1903) owns a sheet plant, formerly the Muskingum Valley Steel Co. The Zanesville plant comprises four roughing mills, six sheet mills, six heating furnaces and nine double-annealing furnaces.

GALVANIZING DEPARTMENT AND FACTORY.—There are four mechanically-fed galvanizing pots at Middletown. These coat the light sheet for sale in the flat or for corrugating or pressing, and also the heavier sheet for corrugated culverts. The factory has machinery for making truss-metal lath, expanded lath, etc., corrugated sheets, rolled corrugated culverts, and pressed siding.

RESEARCH LABORATORY.—A research laboratory is being built at present. This is a two-story building about 40 × 60 ft. It is to contain physical, chemical and electrical laboratories, for development work and for check testing. The routine chemical work will be done as at present in the shop laboratory.

OUTPUT.—The capacity of the works is about 50,000 tons per year. Only two grades of metal are now being produced, the ingot iron here discussed and electrical core metal (transformer and armature sheet).

New Works.

About a mile from the present Middletown works a new and much larger plant is to be built next year. An area of 400 acres has been acquired. It lies 8,000 ft. from the Miami River, with which it must be connected by sewer and water conduit. Wells driven on the property can furnish most of the water required. The equipment will be approximately as follows:

Four 65-ton open-hearth furnaces at first, with room for extension to twelve or more. This, with the present three furnaces, will give a total of seven as soon as the new works are started.

Blooming mill, 40-in. This will have a capacity of about 600 tons of bars in 24 hrs. Slabs of various sizes, and 8-in. or 4-in. billets, will be rolled in this mill. This mill will take large ingots, the intention being to employ top-poured ingots of large size in the new plant. The heat of the ingots will be utilized as far as possible, soaking pits being provided, to eliminate the reheating of cold ingots which must be done in the present plant.

Plate mills. A 96-in. plate mill, capable of rolling plates up to 1½ ins. thick, will be installed. There are to be also two smaller plate mills for work intermediate between the product of the large plate mill and the sheet mills.

Sheet mill. Eight sets of sheet rolls are planned for, with suitable number of heating and annealing furnaces. These mills are to be driven by electric motors, while the heavy mills are to be driven by steam engines. The exhaust from the latter is to be utilized in low-pressure steam turbines driving electric generators to furnish current for the sheet mills and for other electrical use around the plant.

The estimated cost of the new plant is $3,000,000, which has already been raised by a stock issue. Ground is to be broken in two months, and the company hopes that the plant will be completed by the beginning of 1911.

THE TRACK ELEVATION at Grand Crossing, Chicago, has been decided upon as far as the general plans are concerned, and a board of arbitration is to settle the apportionment of costs between the railways involved. This board is to be composed of five members, four of whom have been named, as follows, and these will select the fifth member. E. W. McKenna, Vice-President of the Chicago, Milwaukee & St. Paul Ry.; A. W. Sullivan, General Manager, Missouri Pacific Ry.; J. W. Kendrick, Vice-President of the Atchison, Topeka & Santa Fe Ry., and Darius Miller, Vice-President of the Chicago, Burlington & Quincy R. R. The estimated cost of the work is about $8,000,000.

The work involves special difficulties from the fact that there is a complicated crossing of tracks at the street level, and grades of the tracks are to be separated so as to eliminate the track crossing as well as the street crossing. This necessitates a double deck arrangement, with long approaches. There is a heavy suburban traffic on some of the lines, in addition to heavy main line traffic of passenger and freight trains. In addition to this, electric street railways have to be provided for.

The present plan includes raising the tracks of the Illinois Central Ry. 8½ ft. above datum, and lowering the street grades sufficiently to give a clear headway of 13 ft. in the subways. The parallel tracks of the Lake Shore & Michigan Southern Ry. and Pittsburg, Fort Wayne & Chicago Ry. cross the Illinois Central R. R. at an angle of about 70°, and these will be raised 36 ft. above datum or 24½ ft. above the street with a clearance of 17 ft. above the Illinois Central R. R. The New York, Chicago & St. Louis R. R. tracks from the south will parallel the Illinois Central R. R. on the west side and cross under the other two roads, after which it will turn westward and parallel Lake Shore & Michigan Southern Ry. tracks, being north of these tracks. Provision will be made for six tracks for each railway, necessitating very wide structures.

The Construction of the Substructure of the McKinley Bridge Across the Mississippi River at St. Louis, Mo.

By F. E. WASHBURN,* C. E.

A new bridge across the Mississippi River at St. Louis, popularly known as the McKinley Bridge, is being built for the interurban electric railway lines of the Illinois Traction System by a company organized for the purpose under the name of the St. Louis Electric Bridge Co. Mr. Ralph Modjeski, M. Am. Soc. C. E., is Chief Engineer of the company, and the writer is Resident Engineer in charge of the work.

The bridge (with its approaches) extends from the northern part of St. Louis to the town of Venice, Ill., and is located between the existing Eads and Merchants bridges. It has a double track railway between the trusses and two roadways outside of the trusses. The bridge proper has eight spans, and the general arrangement of the spans and piers is shown in Fig. 1. The channel is crossed by three through-truss spans, 523 ft. c. to c. of piers for the center span, and 521 ft. for the two side spans. At each end is a 250-ft. deck truss span, followed by one 150-ft. deck truss span at the east end, and two 150-ft. deck truss spans at the west end. The approaches are steel viaducts.†

River Conditions.

The Mississippi River at the bridge site is about 1,950 ft. wide at low-water stage and nearly double this width at high water. The west bank has the steeper incline, beginning at low-water shore line near the west inner harbor line and reaching above the high water line at the embankment of the C., B. & Q. Ry. in a distance of 300 ft. The east bank rises with a gradual slope to an elevation of about 15 ft. below the high water line near the east inner harbor line. From this point east to Main St., in Venice, the country is low and flat, and is traversed by sloughs across which railway embankments have been thrown. The main flooded area is on the eastern shore, and the level of this area fluctuates yearly according to the action of the current.

The river bed is composed of sand which is continually changing its level, but in general its slope is towards the east. For a distance of 600 ft. east of the west inner harbor line it is comparatively level at an elevation of from 1 ft. to 4 ft. below the low water line. It then slopes gradually to the east, reaching its lowest point about 400 ft. west of the east outer harbor line, where it is from 18 ft. to 25 ft. below low water line. The changes noted in the river bed during the construction of the bridge were such as to transfer the natural channel of the river toward the east.

The location of the piers was governed by the harbor lines adopted by the War Department in 1903. The distance is 1,500 ft. between the outer harbor lines and 2,000 ft. between the inner lines, thus making a levee area 250 ft. wide on each side of the river. The plans approved by the War Department provided for clear spans of not less than 500 ft. at low water between the outer harbor lines. This required the location of the outer piers, outside of these lines, and the lengths of the main spans were made as above stated. The difference in distance between centers of piers is on account of the larger size of the inner piers. The actual distance from the outer harbor line to the center of the adjacent pier was made 32 ft. 6 ins.

The War Department required a clearance of 50 ft. between high water and the lowest point of the superstructure. The assumed high water level is that of June, 1903, when a stage of 38 ft. was reached; this was the highest stage since 1844, when 41.32 ft. was recorded by the Mississippi River Commission. All elevations used on this bridge were referred to a datum 100 ft. below the St. Louis directrix which marks the elevation of high water in 1828. Check levels were run over the Merchants' Bridge between St. Louis city bench marks and the U. S. bench-mark No. 8 established at the levee in Venice, Ill., from the Memphis datum. These levels were checked across the new bridge piers as soon as construction permitted. The low water stage, that of 1863, coincides with the zero of the St. Louis gage, which is 33.7 ft. below the city directrix, or at an elevation of 66.3 ft. referred to the bridge datum. The elevation of assumed high water was therefore 106.1, allowing for a slope of the water surface of 0.6 ft. per mile.

Probably on account of the banking up of the water at the Eads and Merchants' bridges the slope of the water surface between these structures varies with the rising or falling stage of the river for stages of 20 ft. and above. During the construction of the McKinley Bridge the slope varied from 0.17 ft. per mile at a 5-ft. stage to 0.83 at a 35-ft. stage. The intermediate slopes were as given in Table I. Stages of the rising river were noted from the middle of February, 1908, to the latter part of June. During this time the fluctuations were quite pronounced, and the average rate of rise was about 8 ins. in 24 hours. Stages of the falling river were noted between the latter part of June, 1908, and the early part of September, when there were no pronounced fluctuations, the general rate of fall being about half the rate of rise.

TABLE I.—SLOPE OF RIVER SURFACE BETWEEN THE MERCHANTS AND EADS BRIDGES.

Stage.....	5ft.	10ft.	15ft.	20ft.	25ft.	30ft.	35ft.
	Slope in ft. per mile						
Rising .....0.17 ft.	0.20	0.23	0.45	0.60	0.70	0.83	
Falling .....0.17 ft.	0.20	0.23	0.37	0.40	0.60		

Piers.

The substructure for the bridge proper consists of nine piers. The four piers of the channel spans (Nos. 1 to 4) are of concrete faced with stone masonry. Piers (A), (B) and (C), on the west side and (D) and (E) on the east side, are of concrete. All have caisson foundations, except that the three piers (A), (B) and (E) were founded in open excavation upon wooden piles.

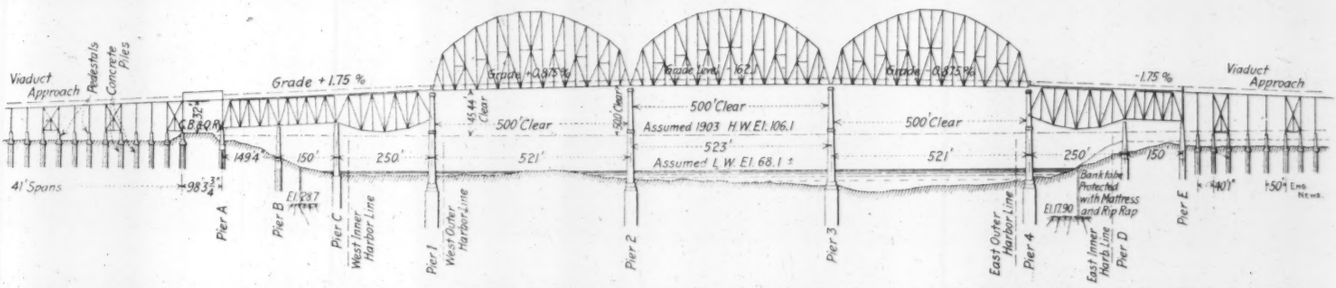


FIG. 1. PROFILE OF RIVER BED AND ARRANGEMENT OF SPANS OF THE McKINLEY BRIDGE OVER THE MISSISSIPPI RIVER AT ST. LOUIS.

The caisson foundations were placed on bed rock, which consists of a bluish gray crystalline limestone practically free from fissures, clay pockets or vertical seams. Horizontal clay seams were found in some cases near the surface of the rock. The surface in general was corrugated and worn smooth by erosion. The corrugations invariably lie in the direction of flow, the greatest span and depth of the valleys being about 1 ft. The material overlying the bed rock consisted of clay in small quantities, quicksand, gravel, sand and small boulders. Its average depth was 30 ft. at the main piers and 40 to 70 ft. at the approach piers. The rock was comparatively level at the sites of the piers, but its general slope was to the east. The difference in elevation across the river was 14 ft., the average slope being 0.7% and the steepest 1.25%. The slope east and west of the main piers averages about 0.5%.

From two to four wash borings were taken at the site of each pier for the determination of the elevation of bed rock. The difference between the average results for each pier and the actual elevations varied from 6 ins. to 8 ft., the average difference being about 3 ft. These results show that little dependence can be placed on the accuracy of wash borings, especially where gravel or boulders are encountered.

The facing of the main piers (Nos. 1 to 4) is of Bedford limestone, except that the bridge seats and the upstream nose stones above the river bed are of granite. Concrete was used for backing, except for the three courses below the main coping course, which are backed with limestone. The curved surfaces of the upstream starlings are close pointed to ¼-in. projection. The exposed surfaces of the main copings and the projecting bottom beds of the belting courses are planed. A 4-in. draft line is cut along the lower edges of the belting courses and on each side of the vertical angles of the downstream starlings. All other stones are quarry faced, with projections not exceeding 3 ins.

The design of the channel piers is shown in Fig. 2, and Fig. 3 is a view of one of the completed piers. A distinctive feature in the design of the starling coping consists in the manner of closure of the pointed and circular portions. The usual starling coping is dispensed with, and a heavy batter is made from the point of the nose at the base of the starling course to the top of the next course above. This results in a conical surface which adds considerably to the appearance of the pier.

Piers 1 to 4 are 10 ft. and Piers 2 and 3 are

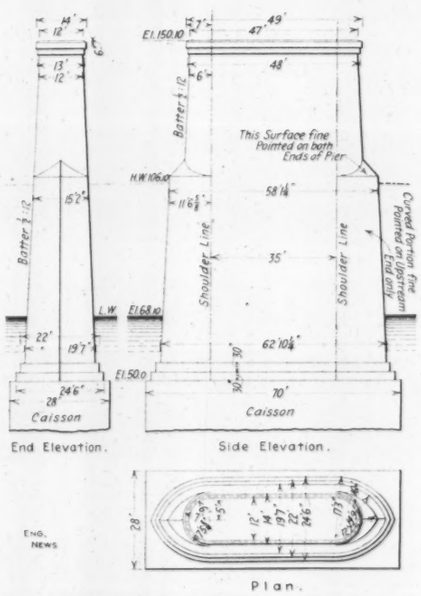


Fig. 2. Piers Nos. 2 and 3 for the Channel Spans of the McKinley Bridge.

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†The superstructure of the bridge will be described more fully in a later issue.—Ed.

12 ft. in width beneath the belting course, with shoulder distances 32 and 35 ft. respectively. Piers A to E are 7 ft. wide beneath the coping, which is 30 ins. high with a 12-in. projection. These piers are reinforced with steel bars spaced 18 ins. apart 6 ins. from the surface in shafts and copings.

Pier B consists of two square pillars on a common foundation with an arched connection at the top; the latter was used for appearance and sta-

Fig. 3. Completed Pier No. 3; McKinley Bridge.
(This shows the upstream end; the starling of the downstream end is of quarry-faced stones.)

bility, the former for the prevention of unequal settlement in the arch masonry. Piers C and D each consist of one monolith with parallel sides and circular ends. Piers A and E are each composed of two square pillars on separate foundations. The batter for all the piers was made ½-in. per ft.

Surveys for Locating the Piers.

The piers were located by means of a triangulation system (Fig. 4), the lengths and positions of the base lines of which were determined by local conditions, which were far from ideal. The C., B. & Q. Ry. occupied the only available position for a base line on the Missouri shore, and the only advantage of this location was its elevation above high water. The disadvantages were the impossibility of using triangulation houses, interruptions due to passing trains, and heat vibrations over a cinder bed. The length of this base line was 2,016 ft., about 66% being to the north of the bridge tangent on account of local conditions.

Collapsible targets were substituted for triangulation houses, each target consisting of a hinged tripod with adjustable target and plumb bob attachment. Hubs with grooved metal tops, in which screws were placed for level adjustment, were driven in the ground for the reception of the tripod tips. This arrangement enabled the target to be accurately set without great loss of time, and was particularly valuable when trains were frequent.

The location of the Illinois base line was made such that the end hubs could be readily seen from the Missouri base line hubs, and local conditions limited its length to 1,910 ft. It was placed at as high an elevation as the nature of the shore would permit, and the end hubs were thoroughly referenced on account of the danger of disturbance by flood. Owing to the general unevenness of the shore and intervening gullies this line was more difficult of accurate measurement than the Missouri base.

Triangulation houses were used on this base line for the protection of the transit and elevation of targets, which were similar to those used for the collapsible targets. The adjustment of the target was made from within the house by means of hook bolts which extended through posts braced against the roof, the lower ends being in reach of the operator. The target was set on a round base and the bolts were placed at the circumference to allow adjustment at any angle. Two windows, hinged at the top to act as sunshades, were provided on adjoining sides of the house, and the corner posts were made so that a beveled section could be removed from the inside thus allowing an unobstructed range of vision. The base lines were laid out for measurement at 98-ft. points, the tape being supported at quarter points. A 100-ft. Chesterman tape standard at 38.1° F and 12 lbs. tension (continuously supported) was used for determining lengths. The temperatures were obtained by means of thermometers with exposed bulbs placed on supports at each end and center of the tape.

The observed values were corrected for sag, temperature and difference of elevation. Observations were made during the months of October and November and the range of temperatures was from 32° to 83° F. Of 31 measurements, 10 were rejected on account of poor weather conditions and the personal equation of observers. The following results were obtained:

	East.	West.
Max. variation in observed lengths.	0.671 ft.	0.287 ft.
Max. variation from mean absolute length	0.0361	0.0284
Max. variation in absolute length.	0.0626	0.0545
Probable error of the mean	1 in 500,000	1 in 590,000
Probable error of a single observation	1 in 150,000	1 in 185,000

These probable errors denote the relative accuracy in the measurement of the two base lines, but as these errors are a function of the variations of the absolute lengths from the mean, the discrepancies in mean absolute lengths are given as being an index of the probable actual maximum error of the work.

The instrument used for angle observations was a Berger & Sons inverting transit reading to 20 secs. of arc. Each set of observations consisted of at least ten settings with the telescope direct and ten reversed, the repetitions being made in opposite directions to eliminate errors of graduation, of clamp and axis movements, and of adjustment for collimation. The target used was a flat board 8 ins. wide and 30 ins. high. The face of the target was divided into triangles, except for a narrow vertical area through the center which was of sufficient width to subtend 4 secs. of arc for a radius equal to the greatest length of sight in the quadrilateral. This angular width was sufficiently great to extend beyond the vertical cross-hair and allow immediate accurate setting of the instrument. This narrow area and alternate triangles were painted white, the remainder of the surface being red. The white triangles merging into the central white area formed an optical line which was easily located.

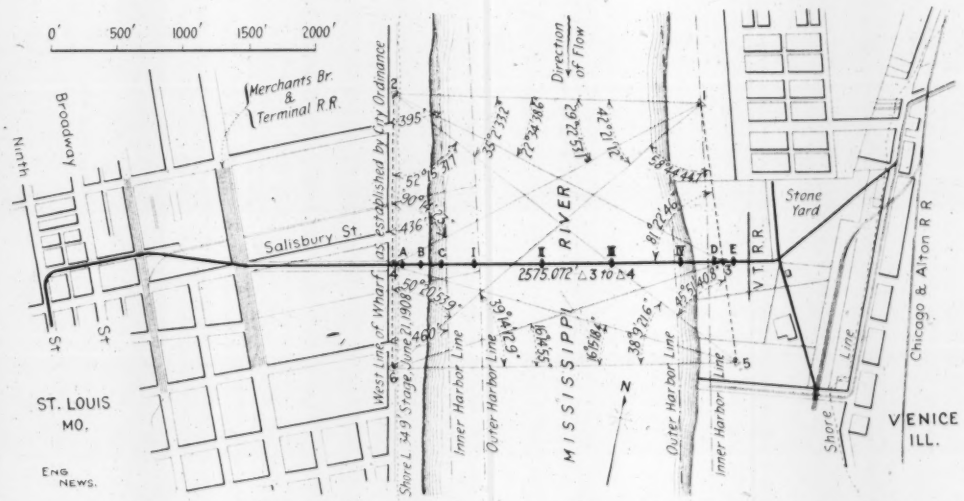


FIG. 4. PLAN OF THE TRIANGULATION SYSTEM FOR LOCATING THE PIERS OF THE McKINLEY BRIDGE.

The angles of the system varied from 35° to 58°. Small angles were avoided on account of the greater error in resulting distances for an equal error in observed values. The observed values of the angles for the entire quadrilateral were corrected according to the angle equation, and these corrected values were again corrected for the side equation according to the tabular differences for one second for the logarithmic sines of the various angles. The angular values resulting from this method of rigid adjustment were used in all computations for pier location after being verified by computation of base line distances. The coordinates of the triangulation stations and piers and the bearings and distances were then computed and tabulated, the bridge tangent being taken as an east and west line for convenience.

Caissons.

The inside width of the caissons was determined by the width of the required bearing area of the superstructure, the height and batter of the pier, and the allowance for offsets in foundation courses. The inside length was determined by the width between trusses and the width and radius of the bottom foundation course.

The height of the caisson proper was dependent upon the required height of the working chamber and the depth of roof necessary for sufficient strength during sinking. The limiting factors for the determination of the height of crib were the elevations of bedrock and river bed, depth of water at the working stage, effective sinking weight, and the relative cost of caisson and masonry construction. For Pier C, the effective sinking weight was the determining factor, and it was found necessary to increase this weight by flooding the cofferdam. In the case of Pier D the height could have been reduced by 15 ft. with good results. The dimensions of the caissons are given in Table II. The depths of water at low water, given in the table, are those during construction. The depths change on account of scour.

A cofferdam was used for each pier, its height varying to accommodate the work for the stage of the river at which the pier was being constructed. This height in most cases was sufficient to allow from 6 to 10 ft. penetration of the cutting edge at the setting of the last offset course. The heights were as follows: Pier No. 1, 12 ft.; No. 2, 36 ft.; No. 3, 10 ft.; No. 4, 6 ft.; C., 18 ft.; D., 4 ft.

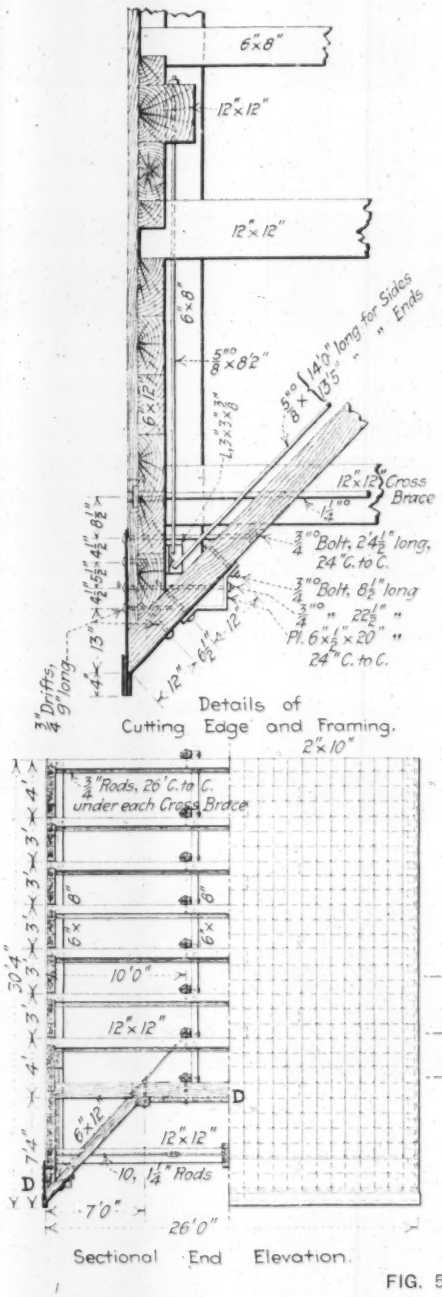
The caissons were of timber (long-leaf pine) and reinforced-concrete construction, and the caisson for Pier 4 is shown in Fig. 5. The concrete was of gravel, in proportions of 1:2½:5, and the reinforcement consisted of ⅝-in. and ½-in. round steel rods. In general the walls were made of 6 × 12-in. timbers supported by 6 × 8-in. and 10 × 10-in. cross and longitudinal struts, making 11 ft. to 12 ft. panels and 3 ft. to 4 ft. vertical intervals. The wall timbers were laid horizontally, with intermediate butt joints and halved corner joints, and they were bound

TABLE II.—CAISSONS AND FOUNDATIONS OF McKINLEY BRIDGE.

Pier.	Depth of water at L. W., ft.	Caisson, outside dimensions, ft.	Height of caisson and crib. ft.	ins.	Relation of weight to penetration—Wt. per sq. ft. of caisson surface exposed to friction. lbs.	lbs.	Depth of penetration, ft.	Sinkage in 24 hrs., Max., ft.	Ave., ft.	Depth from L. W. to bottom of foundation, ft.
1.....	8	26 × 65	20	4	1,850 to	1,145	10 to 33	3.0	1.8	41.27
2.....	17	28 × 70	24	4	3,056 to	1,142	10 to 33	3.0	1.0	47.72
3.....	29	28 × 70	26	4	1,977 to	956	10 to 34	4.0	1.6	59.90
4.....	21	26 × 65	30	4	1,725 to	896	10 to 36	4.3	2.1	52.87
C.....	3	18 × 42	36	4	1,447 to	323	10 to 39	7.3	2.5	39.90
D.....Land		18 × 42	66	4	992 to	898	10 to 70	7.7	4.1	54.00

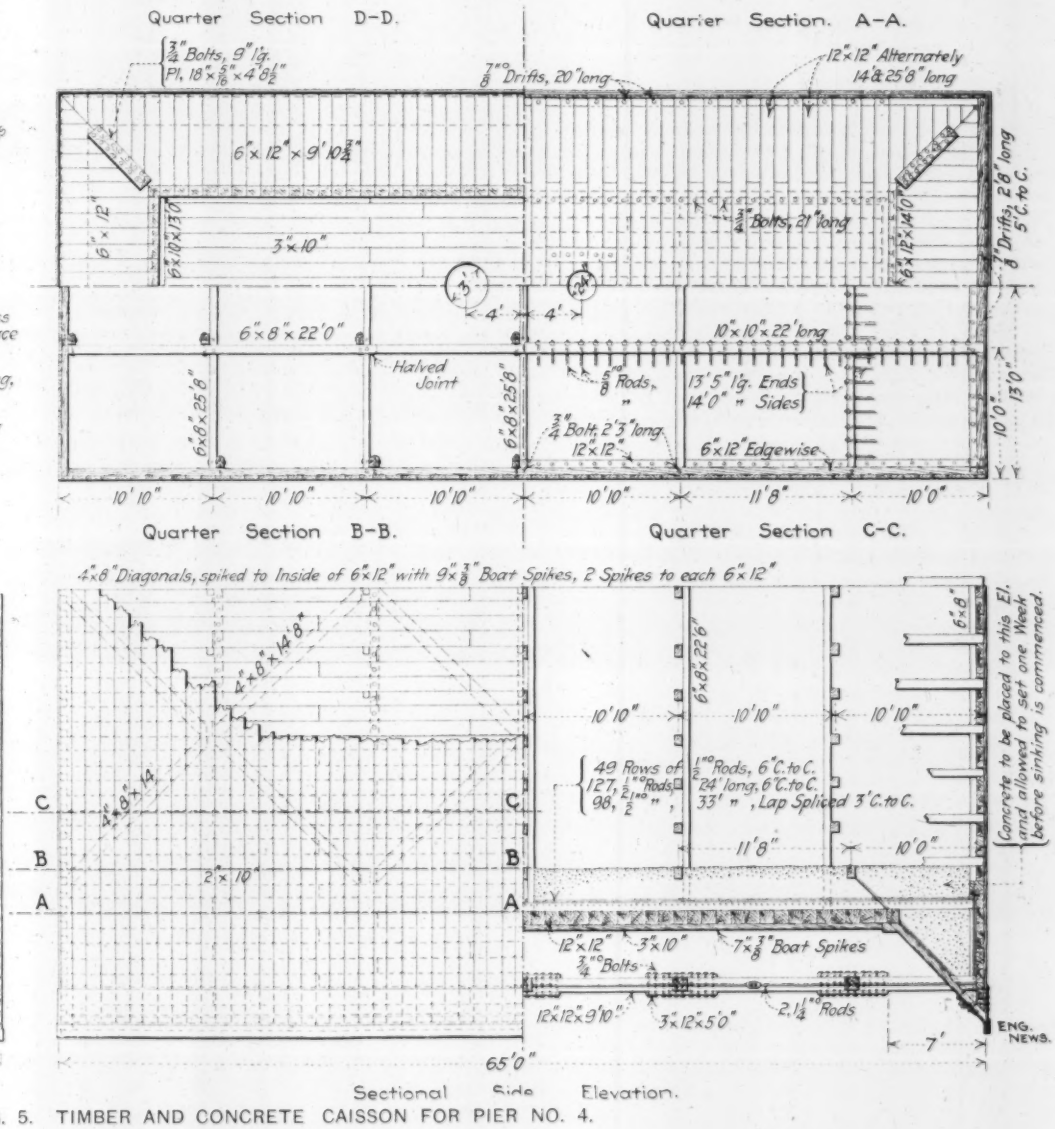
to the adjoining courses by ⅞-in. drift bolts 2 ft. 8 ins. long, spaced 5 ft. in every course. The walls of the cribs in the higher caissons were reinforced by 4 × 8-in. diagonals between the posts.

Posts 6x8 ins. were provided at panel points in the walls and intersection points of the struts.



Details of Cutting Edge and Framing.

Sectional End Elevation.



Sectional Side Elevation.

FIG. 5. TIMBER AND CONCRETE CAISSON FOR PIER NO. 4.

The wall posts were beveled at the bottom where they encountered the inclined roof timbers, while the posts at intersections of struts extended to the horizontal roof timbers. These posts were bolted to the longitudinal struts, which in turn were bolted to the cross struts at intersections. A ¾-in. tension rod was supplied under each cross brace to overcome any tendency to distortion during launching and to counteract any excess of pressure due to fresh concrete. The walls were sheeted with 2x12-in. friction plank laid vertically and thoroughly calked.

The working chamber was made 7 ft. high, with side and end roof timbers sloping at 45°. The main part of the roof was composed of a single course of 12x12-in. cross timbers laid horizontally, and covered with concrete. Each alternate timber extended to the side walls, where it was framed in with halved joints. The intermediate timbers extended a sufficient distance from the center to form butting surfaces for the upper ends of the inclined roof timbers, which were composed of 6x12-in. timbers laid flat. The inclined roof timbers in the corners were necessarily of varying lengths and their upper ends were secured against upward pressure by bent plates bolted on the outside of the roof.

The inclined roof timbers extended to the outside of the sheeting, the lower wall timbers being framed to fit the angle (45°) forming the apex or cutting edge. This edge was fitted with a shoe of ⅜-in. plate. It consisted of two outside plates 33 ins. and 8 ins. wide, riveted at the bottom to a bent plate 3 ft. wide extending up the under side of the inclined roof timbers. The riveted portion of these three plates extending below the timber formed the bottom of the cutting edge, which was 4 ins. high and 1⅛ ins. wide. These plates were spliced at the centers of the sides of the caisson and reinforced at the corners with ⅜-in. plates. The shoe was fastened to the caisson timber by drift bolts at the bottom, and secured at the top by ¾-in. bolts 2 ft. apart extending through the wall and inclined roof.

On account of the liability of buckling of the cutting edge due to the small area at the bottom, it was necessary to provide lugs upon which to distribute the weight of the caisson and pier during construction. These are ½-in. steel plates 6 × 20 ins. spaced 2 ft. apart and bent so as to form horizontal bearing areas of 36 sq. ins. about 16 ins. above the cutting edge. They were drift bolted to the inclined roof timbers at the bottom and bolted at the top, the center being fastened by bolts extending through the wall, sheeting and outside plate. These are shown on the enlarged detail in Fig. 5.

As a further precaution against distortion of the cutting edge, the concrete in the V-shaped portion of the caisson was reinforced by two rows of ⅝-in. round steel rods spaced about 1 ft. apart. One row was placed vertically 2 ins. from the wall timbers and the other the same distance from the inclined roof timbers, the object being to take up the tension to which the concrete would be subjected by distorting strains in either direction. Lug angles bolted to the wall in the apex of the V-shaped part of the caisson were used to hold the lower hooked ends of the rods in position while concrete was being deposited. The upper threaded ends of the vertical rods were held in position by being extended through projecting wall timbers in the second course above the roof, while the upper ends of the inclined rods were bolted to stay rod supports in the third and fourth courses.

The walls of the chamber were reinforced by one longitudinal brace and five cross braces, (all 12x12 ins.), which extended through the inclined roof timbers and butted against the walls between the second and third courses above the cutting edge. On either side of each brace a 1¼-in. round steel rod extended through the caisson, the washers at the ends being countersunk

in the wall timbers. These rods were provided with turnbuckles by which they were adjusted to give sufficient tension to prevent the distortion of the walls. The ceiling of the chamber was planked with 3 × 12-in. sheeting laid longitudinally and 6 × 10-in. waling timbers were provided in the angle of the roof, these timbers being beveled to receive the upper ends of the inclined roof timbers. The entire inner surface of the chamber was thoroughly calked with oakum and all joints in the iron plates were coated with asphaltic cement to prevent leakage of air through the walls.

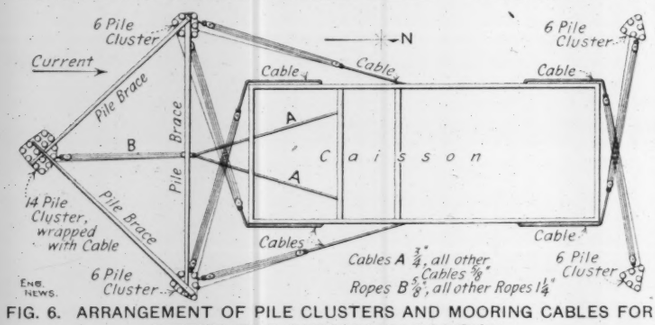


FIG. 6. ARRANGEMENT OF PILE CLUSTERS AND MOORING CABLES FOR SETTING THE CAISSONS IN POSITION.

The roof or deck consisted of the single course of 12 × 12-in. timbers and a covering of reinforced concrete. The steel reinforcement for the latter consisted of two layers of ½-in. round steel rods placed longitudinally and transversely from wall to wall. The rods were spaced 6 ins. c. to c. and the layers were placed as close to the deck and as close to each other as good bonding would allow.

The above description applies in general to the caissons for Piers 1, 2 and 4. On account of the assumed greater depth of water at Pier 3 the caisson for that pier was made considerably heavier than the others. The walls and posts were made of 12 × 12-in. timbers, and the cross and longitudinal braces were similar timbers framed into the walls with dovetailed joints. The tension rods in the cribs were omitted, while the splicing of the struts and the details in general were made stronger than in the other caissons.

The construction of caissons for Piers C and D was similar to that for Piers 1, 2 and 4, except as to walls and bracing. The solid wall in the caisson for Pier C extended 24 ft. 4 ins. above the cutting edge, the remaining 12 ft. of the wall of crib being composed of 2 × 12-in. sheeting on 6 × 12-in. timbers supported by 6 × 6 in. stay blocks at 4 ft. vertical intervals. Toe braces of 6 × 8-in. timbers were supplied in the end panels. The solid wall in the caisson for Pier D extended 11 ft. 10 ins. above the cutting edge; the remaining 54 ft. 6 ins. of the wall and the bracing, etc., were similar to that for Pier C.

Timber and Concrete Caissons.

The type of caisson originally intended for this bridge was of timber, similar to that used on the Thebes Bridge, but the type actually used was designed by the Missouri Valley Bridge & Iron Co., the contractor for the substructure. This design was modified by the engineer only in the case of the caisson for Pier 3.

This style of caisson differs from the ordinary timber caisson chiefly in the use of reinforced concrete in the place of timber in the construction of the walls and roof. The timber roof was designed to support only a few feet of fresh concrete and this safe depth was the limit set for concreting until an interval of one week had elapsed. Further increments of weight were then supported by the reinforced-concrete deck, the transverse reinforcement being made sufficient to increase the strength of the roof to an amount necessary to bring the angle of loading to 60° and to develop a constant working stress in the steel rods for all increased loading.

The longitudinal reinforcement was made sufficient to prevent distortion in a horizontal plane, and to distribute the weight. In the actual conduct of the work it was not found necessary to delay construction for setting of the concrete when the safe load had been placed on the timber deck, as the conditions of support of the cutting edge for which the caisson was designed were encountered only at Pier D. There the roof was made comparatively much stronger than in the larger caissons, and, therefore, work was carried on continuously as in the others. This caisson was designed for construction at the site of the pier, and the roof was not designed to resist upward pressure. It became necessary, therefore, to concrete the V-shaped portion of the caisson before launching in order to prevent the displacement of the roof timbers at the waling and the dislodgement of calking by the pressure of the confined air during launching.

The economy in the use of this type of caisson lies in the saving of timber and the time and labor of framing. The ratio of timber volume to total volume of a timber caisson 28 × 76 × 20 ft. was 37.5%, and the same percentage for a caisson of reinforced-concrete construction 28 × 70 × 24 ft. was 13.2%. For caissons of comparatively equal size this represents a saving of 64.8% of the timber volume, and a substantial cost reduction varying with the relative prices of timber and concrete construction. The iron cutting edge of this caisson has an advantage over the wooden one in that it sinks more readily and its form is such that it is more easily reached in excavating. Its chief disadvantage is in its comparatively small bearing area, which necessitates the use of bearing blocks for uniform distribution of the weight to prevent buckling during construction.

Launching and Placing the Caissons.

Each caisson was built on shoes or butterboards bolted to launching ways which were supported on pile foundations with a slope of 1¾ ins. per ft. These shoes consisted of timbers extending the full width of the caisson, resting on caps upon which they slid during launching. They were spaced about 6 ft. apart and were bolted to the caps on the land side of the caisson. The launching was started by sawing the shoes simultaneously below these bolts and thus allowing the caisson to ride to the water. The launching ways were extended into the water a sufficient distance to allow an immersion of about 10 ft. In order to reduce the required depth for immersion the caisson was only partially constructed preliminary to launching. The timber work was completed as required for concrete construction in order to reduce the cost of handling the material.

Before the caisson was launched, the warp of the cutting edge was determined by levels to the four corners and its plane was established through three of these points. Benches with known equations to this plane were placed at the corners above the depth of immersion and perpendiculars to the plane at the centers of the sides and ends of the caisson were extended by striking equal arcs of circles from the established corners. By this means the position of the cutting edge could always be determined, irrespective of the irregularities in timber construction.

Four pile clusters were located by triangulation and driven near the proposed site of the pier. The largest cluster, usually consisting of about 14 piles, was driven at a point on center line of pier about 35 ft. north of the north end of the caisson, and a smaller cluster was driven at a point about 15 ft. northeast of the northeast corner. The other two small clusters were located at corresponding points off the northwest and southwest corners. All clusters were bound with cable, and those to the north were braced together at the top by piling placed horizontally, forming a triangular support between the clusters. After the caisson was towed to position, a fifth cluster was driven off the southeast corner. Fig. 6 shows the arrangement of the pile clusters and mooring lines.

It was necessary to moor the caisson in such a manner as to allow freedom of movement and at the same time to secure stability. This was accomplished by the use of cables rigged with block and tackle. The main cable was attached to the large cluster, and two strands were passed through the end of the caisson and fastened at each side to one of the roof timbers. This main cable, and two cables which passed direct from the small north clusters through the sides of the caisson, were used for south movement. A cable from each of the corner clusters passed across the caisson and was fastened to the opposite side about 10 ft. from the corner. These four cables were used for lateral and skew adjustment.

The caisson was located approximately in position before concreting was commenced, but was not located accurately until after the cutting edge struck the river bed. An excess weight of about 100 tons was then added and air pressure applied sufficient to raise the cutting edge free from the river bed while the caisson was being located. Locations were made usually by simultaneous observations from opposite ends of the same base line. Small platforms were built at the ends of the caisson upon which the observers

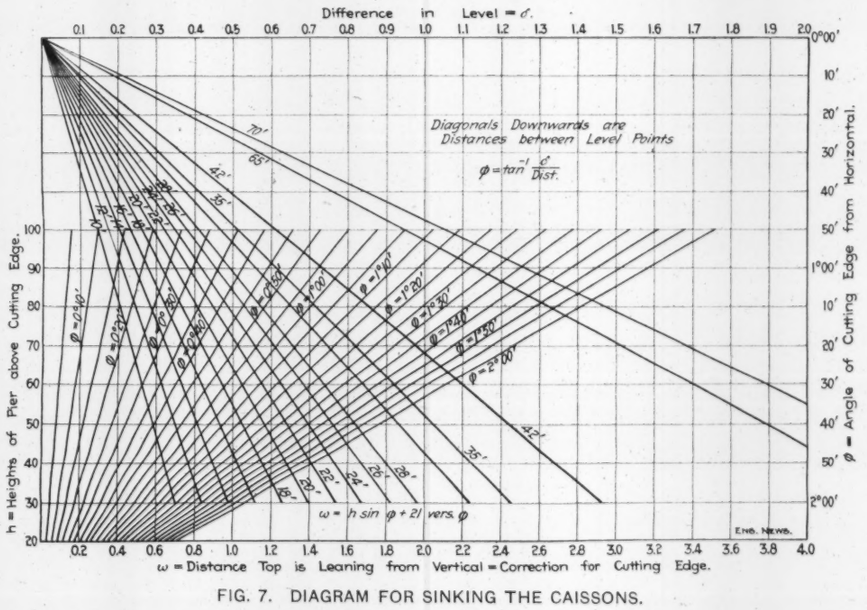

Difference in Level = d.
Diagonals Downwards are Distances between Level Points
φ = tan⁻¹ d / Dist.
h = Heights of Pier above Cutting Edge.
φ = Angle of Cutting Edge from Horizontal.
ω = h sin φ + 2 l vers φ
ω = Distance Top is Leaning from Vertical = Correction for Cutting Edge.
ENG. NEWS.

FIG. 7. DIAGRAM FOR SINKING THE CAISSONS.

located the exact points. A comparison of these located points with the corresponding points on the cutting edge referred to the elevation of the observed points readily denoted the movement necessary to bring the caisson to its correct position. The required movement was made by the use of the cables above mentioned and the operations of location, comparison and movement were repeated until the correct position was obtained. Then the air pressure was gradually released and the caisson allowed to settle in the mud.

Excavation was begun as soon as possible after the location was made in order to prevent possible distortion of the chamber struts and to make the position more secure by the penetration of the cutting edge. The mooring cables were removed as soon as this penetration was sufficient to render the caisson safe from disturbance by the direct effects of the current. Difficulty was experienced in preventing the caisson from moving against the current during early sinking on account of the comparatively short length of the cables, and this is probably the only point in which this method of location is inferior to that in which long cables and anchors are used. As sinking proceeded it was necessary to determine the position of the pier each day when atmospheric conditions permitted. This consisted in determining the levels of the cutting edge by means of the established points on the caisson previously described (or to points transferred to the shoulder points of the pier as construction required) and simultaneous readings for position by observations from the base lines.

The point observed was on the bridge tangent at a measured distance from the center of the pier as constructed. This bridge tangent was thrown across the pier for the determination of north and south position and skew. Usually but one transit was used, the observer establishing the bridge tangent and then determining the angle by the repeating method from a base line hub. The hub selected depended upon the distance of the pier from the base line and the resulting value of the observed angle. Any movement of the pier between these transit and level observations was noted by the levelman, who then signaled for the repetition of observations.

In order to facilitate the field computations a diagram (Fig. 7) was made for determining the corrections to observations for level position. The upper part of the diagram consisted of a group of lines constructed on the following formula

$$\phi = \tan \frac{-^1 S}{D}$$

where S is the difference in level and D is the distance between observed points. The lower part of the diagram was constructed on the formula $M = h \sin \phi + 21$ vers. ϕ. Here M is the correction, h is the height of the observed point from the plane of the cutting edge, and ϕ is the angle of lean determined from the upper part of the diagram. The second term of the equation depends upon the distance between the observed points, and as this was a variable for each caisson and for each masonry observation the number 21 was introduced as an average quantity so that the diagram could be used for any observation. This assumption involved a maximum error of $\frac{5}{8}$-in. for the limiting value of ϕ on the diagram. When ϕ exceeded 2° in value the correction was figured for actual conditions.

Sinking the Caissons.

The forces considered in the sinking of the caisson were the weights of the caisson and pier, the buoyancy of the water, the air, water and sand pressures, skin friction, and current with its counteracting effect of eddies. The magnitude of these forces could be definitely determined, with the exception of sand pressures and skin friction.

Ordinarily the effect of the current as a direct disturbing force is negligible, but as an agent of erosion and deposition it becomes a factor of much importance, nullifying entirely its own direct force by the counteracting effects which it produces. This is especially true in the case of a rising river, for the energy of a rapid current impinging on the end of a caisson is expended in scour and consequent reduction of sand pressure. The sand thus removed by erosion is carried in suspension to the sides and opposite end of the caisson, the greater part being deposited at the downstream end where the eddies are produced. The resultant effect of the current, therefore, is to create a tendency to movement opposite or at right angles to the direction of flow and its magnitude depends upon the rate of flow and is equal to the resultant sand pressure.

Daily soundings were taken during the entire period of sinking in order to guard against undermining of the cutting edge and possible loss of the pier by displacement or overturning. Exposure on the cutting edge was especially guarded against during a high and rising stage of the river, for rapid sinking was then required to prevent the erosion from following the cutting edge down and exposing the caisson to the direct force of the current, when displacement was almost certain to follow if the lateral sand pressures were unequal. During a falling stage of the river this danger was not so imminent, on account of deposition.

The value of skin friction for starting purposes was found to range from about 300 lbs. per sq. ft. for 40 ft. depth to about 600 lbs. per sq. ft. for 70 ft. depth of penetration. As the magnitude of frictional resistance increases with repose it is probable that these values should be reduced about 50% for sinking purposes. The average weights per sq. ft. of surface exposed to friction for penetration exceeding 10 ft. for the different piers are given in Table II. On account of the weights made necessary to facilitate construction it was impossible to determine actual frictional values except in those cases where the ratios of net weights to exposed surfaces were not excessive.

The greatest difficulties in sinking were at Pier 2, as described later. The sinking of the caisson for Pier 3 in the fall of 1907 was delayed by floating ice, and the pier had to be abandoned when the chamber was about half filled with concrete. The shafts were capped and an ice flow broke off a section of the material shaft, allowing the chamber to be filled. This necessitated the removal of the mud and thorough cleaning of the concrete before sealing was resumed.

The caisson for Pier D was located 6 ins. east of its proper position, with the expectation that there would be a westward movement due to the pressure of the bank on the east. But the cutting edge actually moved 7½ ins. further east, in opposition to the lean. The water was about 13 ft. below the surface of the ground when sinking was begun and this condition was taken advantage of by ditching the caisson for that depth. This pier was submerged the day that sealing was completed.

The caisson for Pier C was carried away by drift soon after it was moored in position, but it was captured without damage above the Eads Bridge. It was necessary to sink this caisson to bed-rock before the shaft of the pier was located, and the cofferdam was flooded in order to provide sufficient weight for sinking. When an endeavor was made to unwater the cofferdam it was found impossible to do this on account of an opening between the caisson and cofferdam, which was caused by the friction of the sand on the walls of the cofferdam. Concrete was, therefore, deposited in a narrow space between the inside wall of the cofferdam and the outside of the pier, thus forming a concrete cofferdam which was unwatered without difficulty as soon as the concrete had set.

With the exception of the cases above mentioned there was no particular difficulty encountered in the sinking of the six caissons. Table III. shows the maximum variations of the positions of the various caissons from the true positions, the figures referring to the center of the pier in each case:

TABLE III.—VARIATIONS IN POSITIONS OF CAISSONS.

Pier.	Displacement in ft. N.	S.	E.	W.	Total variations in ft. N. & S.	E. & W.
1.....	0.22	0.04	0.21	0.16	0.26	0.37
2.....	0.80	0.40	1.25	0.70	1.20	1.95
3.....	0.00	0.27	0.02	0.28	0.27	0.30
4.....	0.43	0.08	0.58	0.46	0.51	1.04
C.....	0.14	0.05	0.32	0.00	0.19	0.32
D.....	0.17	0.16	1.10	....	0.33	0.64

Pressure and movement curves prepared from daily records gave the approximate conditions existing during sinking, and Fig. 8 shows these curves for Pier 1. An exact record of the variations in movement and position would, of course, be impracticable (if not impossible) on account of the frequent changes in level. The records show that with favorable conditions of penetration and weight the movement of the caisson can be controlled without difficulty. But the movement, especially in sand, does not always follow the law of lean, frequently working directly opposite to that law. The records also show that the general tendency of movement is opposite to the direction of flow, on account of the invariable excess of sand pressure on the downstream surface of the caisson. Fig. 9 is the record of sinking at Pier 1.

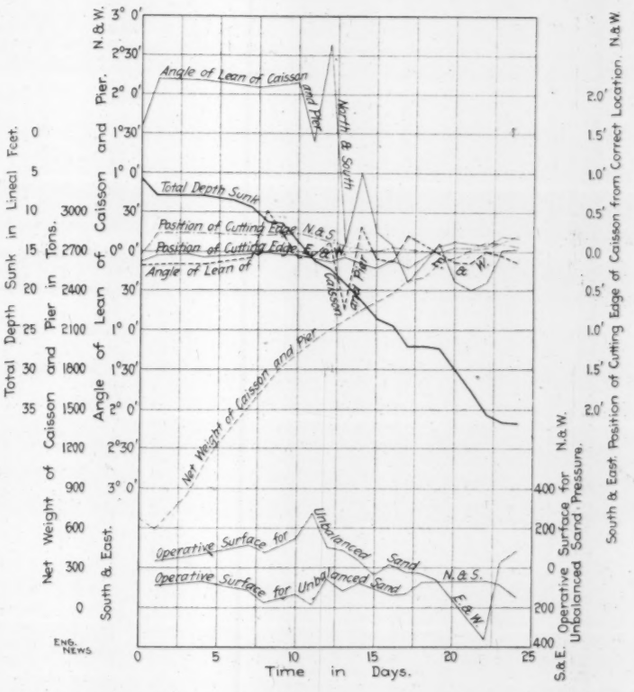

Total Depth Sunk in Lineal Feet.
Total Depth Sunk and Pier in Tons.
Angle of Lean of Caisson and Pier.
Net Weight of Caisson and Pier.
South & East.
N. & W.
Angle of Lean of Caisson and Pier
North & South
Total Depth Sunk
Position of Cutting Edge, N. & S.
Position of Cutting Edge, E. & W.
Angle of Lean of Caisson
Net Weight of Caisson and Pier
Operative Surface for Unbalanced Sand
Operative Surface for Unbalanced Sand
N. & S.
E. & W.
Time in Days.
S. & E. Operative Surface for Unbalanced Sand Pressure.
South & East Position of Cutting Edge of Caisson from Correct Location.
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FIG. 8. PRESSURE AND MOVEMENT CURVES OF THE CAISSON AT PIER NO. 1.

Sinking was done in all cases by means of the wet or dry blow, the latter being used only under low air pressures. There were two discharge pipes and one spoil shaft for each caisson, the latter being used only for excavation in boulders and for the handling of the shoring timbers and riprap. The excavated material was practically uniform throughout, consisting mainly of coarse sand, quicksand, gravel and boulders. Loess and wood were also encountered in the shore piers. Two forms of clay hoist were used (Fig. 10). In one, the bucket was hoisted through the top and swung clear of the pier before dumping. In the

other the bucket remained in the hoist and its contents were dumped through a side door on to a sloping platform from which they were shoveled into the river. Fig. 10 shows the arrangement of the plant for the pneumatic sinking work.

The rate of sinking varied with the size of caisson, depth of water, temperature of air, character of excavated material, number of pressure men employed, and rate of masonry or caisson construction. The average rate for all the piers approximated 2 ft. in 24 hours, and the maximum rate was 7.7 ft. in 24 hrs. In Table II are given the maximum and average rates of sinking, the average rate being obtained by dividing the total sinkage under air pressure by the actual number of days during which sinking was in progress. The number of hours work per day was regulated according to the depth of immersion, the following scale being adhered to:

Immersion, ft.	Hours, No.	Shifts, No.	Immersion, ft.	Hours, No.	Shifts, No.
0—50	8	..	60—70	4	2
50—60	6	2	70—80	2	2

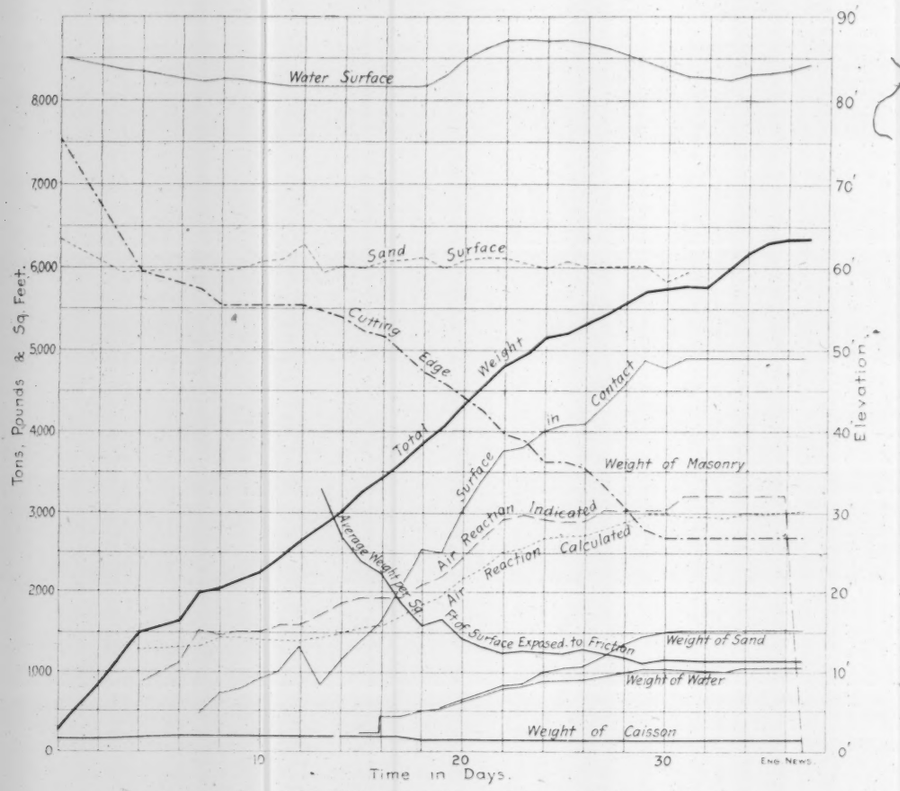


FIG. 9. DIAGRAM OF RECORD OF SINKING THE CAISSON AT PIER NO. 1.

Difficulties in Sinking the Caisson for Pier No. 2.

By far the most interesting sinking was that for Pier 2, where the construction was greatly interfered with by the high water of the summer of 1908. During the progress of sinking, the water rose 21 ft. and the surface velocity of the current increased from 4 ft. per sec. (2.7 mi. per hr.) to 8.8 ft. per sec. (6 mi. per hr.), and this velocity was greatly increased in the immediate vicinity of the pier.

Considerable scour was anticipated and sacks of gravel were thrown in above the caisson before it reached the river bed. As soon as the cutting edge neared the bottom the scour increased rapidly and the sand was piled up by the current in the downstream end, causing the caisson to land on that end first. Launching timbers which had not become disengaged also held up the downstream end and served as a pivot for a skew movement of the caisson. As soon as air was applied these timbers were cut up and removed and the sand which had piled up to the beams in the downstream end of the chamber was sacked and carried to the opposite end for protection of the cutting edge against undermining.

The sand blow was then used in the downstream end to bring the caisson to a level position and sinking was carried on as rapidly as the construction of the masonry work would allow. After sinking about 8 ft. the water rose so rapidly that the masonry work could not be kept in advance of it and the chamber was filled with sand while cofferdam and masonry work was rushed. The increasing pressure on the cofferdam required the addition of toe braces to support the upstream wall, and the lower portions of the cofferdam were flooded as soon as possible after the masonry work was constructed in order to balance the water pressure.

When an apparently safe leeway had been obtained for sinking, the chamber was cleared and shoring was placed to correct the skew, which had increased to 7½ ins. in 70 ft. This skew was entirely corrected in a drop of 3 ft. against an average penetration of from 3 ft. to 5 ft., the net weight moved being from 1,300 to 1,400 tons.

Considerable scour was then observed to the east and the caisson was given an inclination to overcome the tendency to move in that direction, but the easterly movement was rapid, averaging nearly 3 ins. per day for six days during a total sinkage of 10 ft. The maximum movement was 5 ins. in 24 hrs., which occurred at two different times within the six days eastward and northward movement. The eastward displacement was finally arrested by a lean of 2° 38'.

On account of the continued rise in the water, sinking was discontinued and it was not resumed for 8 days, during which time undermining of the cutting edge was prevented by placing sacks of gravel in the chamber and outside of the upstream wall of the caisson. Watchmen were kept constantly in the chamber with a supply of gravel in sacks to replace that drawn out by the force of the current. Sinking was again resumed after building up the masonry, but the gravel which had been deposited outside of the caisson did not seem to protect it from scour. The rate of sinking was not great enough to prevent the scour from following the cutting edge, and after sinking a few feet the chamber was filled with sand to the beams and all attention was given to proper protection of the pier from greater displacement.

The height of the pier had reached 69 ft., with little or no penetration on the upstream end, on account of the gravel which was intended for protection having been carried away by the current. An attempt was made to lower sacks of gravel by means of a clamshell bucket, but this was attended with little success and with great danger of losing the bucket.

Finally the following method was used: Five sections of supply shaft, 2 ft. diameter, were bolted together to form a chute which was suspended from a derrick boom by cables which passed through bolt holes in the flanges of the shaft. Cables fastened to the lower end of the shaft and passing beneath barges moored beside the pier formed a means for controlling the location of the discharge. By this method riprap was placed exactly in the desired position in depths of water exceeding 60 ft. and in a current exceeding 6 mi. per hr.

Accurate soundings were taken through this shaft and the riprapping was continued until a penetration of from 20 to 23 ft. had been obtained. The pier remained in this condition until the flood had abated, the water rising to within 0.1 ft. of the top of the masonry before starting to fall.

Air pressure was then taken off until sufficient leeway for sinking had been obtained, and the caisson was given a heavy lean to the north to correct the north displacement. Shores with a batter of 1 in 12 were placed in the north end of the chamber to correct the skew of about 16 ins. to the northeast. A dredge was employed to reduce the pressure on the west side of the pier, in addition to the usual method of decreasing the pressure by pulling in from the chamber.

The shores used were timbers 12 × 12, 10 × 10 and 8 × 8 ins., resting on timber grillages, but considerable difficulty was experienced in preparing suitable bases below the cutting edge on account of the interference by water. The entire northern half of the chamber was filled with these struts set on a batter in the direction of the desired movement. The net weight of the pier during this work was 3,750 tons and the penetration was from 25 to 15 ft.

Probably on account of the excessive weight and penetration the results were unsatisfactory in the correction of skew. As a result of a heavy lean of 3° 36' the north displacement was corrected an amount of 3½ ins. and a lean to the east improved the east displacement by 6 ins. The remainder of the errors of position were corrected in the batter and level of the pier, and the skew of the belting and main coping courses.

Building the Pier Foundation.

Upon the completion of sinking, the bedrock was tested by means of drilling, and the surface was thoroughly cleaned preliminary to sealing. The latter operation was carried on by benching methods, the material being concrete (1:2½:5) and mortar. In order to obtain a compact body of concrete in the chamber the mixture was made dry and tamped in layers about 10 ins. thick. These layers were begun about 6 to 8 ft. wide at the cutting edge and they were worked up in benches gradually narrowing in width toward the top. A space of 4 ins. was left under the ceiling and struts; into these mortar was rammed as each strut was reached and as each bench was carried to the roof. On account of the rapidity with which the concrete set there was no difficulty in keeping the benches in good order.

The concrete was dumped from the bucket through a funnel into the upper section of the material shaft. The funnel was then removed, the upper section equalized and the concrete allowed to fall into the chamber, from which it was shoveled to the various benches. The clay hoist was also used for sealing, in which case the concrete was dumped from the bucket to a platform at the door of the hoist. From there it was shoveled on to the steel door at the roof of the chamber and the entire shaft was equalized for the final drop to the chamber. The former method was found to be the more satisfactory, as it economized air and did not require an extra handling of the concrete.

The final trench between the material and main shafts was filled by sacking in concrete and the

main shaft was filled for several feet by the same method, the air being supplied by a reduced extension of the main air pipe from the chamber. Air pressure was then retained for 12 hours, after which the filling of the shafts and pipes was completed under atmospheric pressure.

The rate of sealing depended upon the same conditions as the rate of sinking and the results obtained are given in Table IV. For the greater

TABLE IV.—FILLING THE WORKING CHAMBERS.

Pier.	Depth immersed, ft.	Air pressure indicated, lbs.	Temperature outside, degs. F.	Yardage—Total, cu. yds.	Yardage—Per hour, cu. yds.
1.....	56	26	77	297	3.1
2.....	68	31	90	355	2.0
3.....	53	25	54 & 88	355	2.4
4.....	55	26	63	297	3.2
C.....	51	28	79	110	3.4
D.....	73	37	43	110	3.0

depths more time was lost in changing shifts than in other cases. An average of about one hour per day of each man's time was consumed in transfers through the lock and shaft. During warm weather, the air from the receiver was allowed to pass through immersed pipes before going to the chamber in order to regulate the caisson temperature. By means of a 1-in. ventilation pipe and an electric fan a good working temperature was maintained in nearly all cases. The temperatures given are the average maximum temperatures for the time during which sealing was in progress.

After sealing was completed, the sections of shaft extending above low water were removed where provision had been made for removal by boxing the shaft sections and the entire space was filled with concrete. Masonry work then continued at a more rapid rate owing to the non-interference of shafts, air and water pipes, etc.

Locations of the pier were determined every second or third course, and computations were simplified by the use of a table giving angles to the center of the pier for varying distances east or west of its correct position. The most advantageous baseline station was selected for observations on each pier, and the angles were usually figured for intervals of ¼-in. from 0 in. to 2 ins. in each direction. Where the variations exceeded this amount locations were determined more frequently and the position figured for each particular case.

After the shoulder points were established, offsets for striking the arcs were laid off by the foreman mason and inspector, who also trammeled the starling for error of position of nose point and error in radius of ends. Levels were transferred from benches established in the bedding joints for every second or third course as the work progressed. Small errors in position of the pier were corrected by varying the batter of each course the desired amount. In this manner each pier was completed in the required position at the desired elevation for the bearing of the superstructure.

Concrete Pile Foundations for the Approaches.

The approach foundations were placed in ground alternately wet and dry on account of the fluctuations in river stages, and concrete piling was used therefore in all cases. The piles are of the Chenoweth type, and a plant was installed at the bridge site in St. Louis for their manufacture.* The piles were 14 to 15 ins. diameter in lengths of 25 ft., 20 ft., 16 ft. and 14 ft., capped and pointed with mortar. Their weight averaged about 170 lbs. per lin. ft. They were made of gravel concrete mixed dry in the proportions of 1:2:4, the pebbles ranging in size from ¼-in. to 1¼ ins. The points consisted of conical tips 12 ins. long and 8 ins. diameter at the small end, which were formed on the piles immediately after the rolling was completed. The caps were composed of mortar placed after rolling in order to true the driving head of the piles. The reinforcement consisted of five ⅝-in. round Johnson bars wired to two widths of wire netting (No. 19 wire, ½-in. mesh), 36 ins. wide. No. 17 binding wire was used during rolling to hold the pile in shape.

After seasoning for 30 days or more the piles were driven to an average penetration of ¼-in. by the use of a No. 2 steam hammer weighing 7,000 lbs. and having a 3,000-lb. drop operating through a drop of 2 ft. at an average rate of 70 blows per minute. A special cushion cap was provided to reduce the shattering effect of the blow, and a rope cushion was placed on the head of the pile for the same reason. The special cap consisted of a cylindrical steel shell with an oak bottom, rubber filling and a cypress driving head. After the pile had reached its full penetration below the surface of the ground the cap was placed on a follower which consisted of a cypress driving block fitted with a steel hood which engaged the head of the pile. By this means driving was continued from 4 ft. to 6 ft. below the surface, thus dispensing with preliminary excavation and greatly facilitating the movement of the pile driver.

More than 1,000 piles were driven in this manner without fracture and with as much ease and dispatch as ordinary wooden piles. The weight of the piles (170 lbs. per lin. ft.) increased the labor of handling in comparison with wooden piles, but on the other hand it reduced the difficulty of driving. The soil penetrated was variable in quality and consisted mostly of muck and filled ground near the surface, and silt, sand and clay at greater depths.

*Engineering News, May 20, 1909.

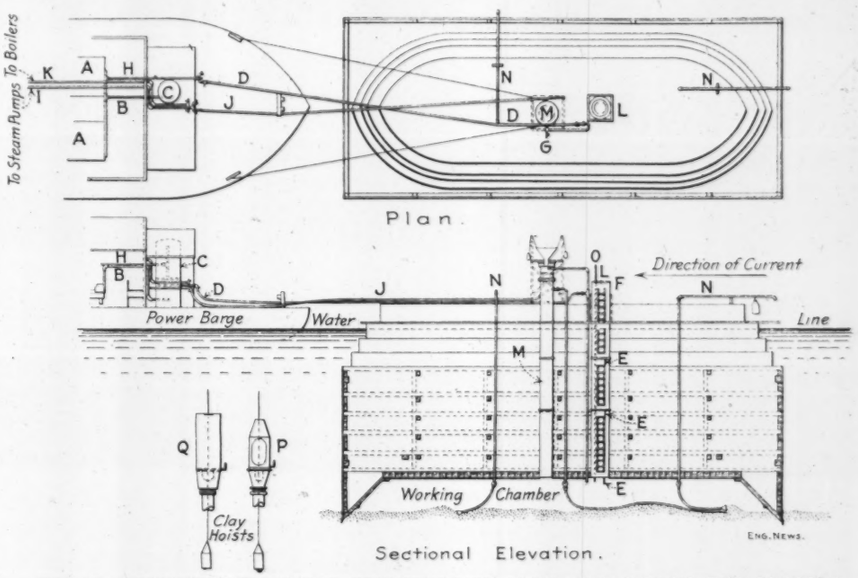


FIG. 10. ARRANGEMENT OF THE CONSTRUCTION PLANT FOR SINKING THE CAISSONS.

A. Air Compressor.
B. Air Delivery Pipe.
C. Air Receiver (48-in.).
D. Air Hose (3-in.).
E. Air Pressure Equalizer Valve (¾-in.).
F. Air Atmospheric Equalizer Valve (2-in.).
G. Air Exhaust.
H. Air Emergency Pipe (4-in.).
I. Water Pipe (3-in.).
J. Water Hose (3-in.).
K. Steam Pipe (2-in.).
L. Main Shaft (36-in.).
M. Material Shaft (24-in.).
N. Sand Blow Pipe (4-in.).
O. Ventilation Pipe (1-in.).
P. and Q. Clay Hoists.

Pedestal Blocks.

The approaches have 41-ft. spans with four columns to each bent; two for the railway and two for the roadway supports. Five concrete piles were used for each railway pedestal and one for each roadway pedestal (12 to each bent). Each pedestal has two foundation blocks 2 ft. thick, the lower one octagonal and the upper one hexagonal. These blocks support a conical frustum shaft, reinforced near the surface with ⅞-in. vertical and ½-in. horizontal square Johnson bars. Anchor bolts, 3 ft. to 4 ft. long were placed simultaneously with the concrete, and adjustment was provided for erection by encasing the bolts in pipes which extended 1 ft. below the surface of the concrete.

The tops of the pedestals are at such an elevation as to keep the steel work above high water.

Construction Plant.

The contractor's yards were located on a strip of ground above ordinary high water, about 600 ft. east of the inner harbor line, near the bridge approach in Venice. An incline trestle was constructed from this point to the river for delivery of material to barges. All of the timber and stone and part of the cement was handled in this manner. The cement was stored in a warehouse near the incline trestle and also in a storehouse at the west end of the Merchants' Bridge. The sand was dredged from the river near the bridge site, and the gravel was shipped by rail from the Meramec River to St. Louis, where it was loaded on barges for delivery at the site. All handling of material in the yards was done with a 10-ton derrick car, and the transfer of material at the river was done with derrick barges. Cement was shot down in a chute from the trestle to barges.

The fleet engaged in the construction consisted of a pressure barge, pile-driver barge, three derrick barges, two concrete barges, four material barges, one cement barge, two coal barges, one signal lighter and one tug. The pressure barge was a model barge 28 × 155 ft., 6½ ft. deep. It was equipped with three marine boilers with a combined capacity of 180 HP.; these were fed by a feed pump, an injector, and an inspirator, the latter being used in case of accident to the feed pump or feed water heater. The boilers supplied steam to two air compressors 10 × 13½ × 14 ins. which delivered compressed air to a receiver from which it was piped to the caissons. Air pipes were also laid for direct delivery in case of an emergency. A steam pump 8 × 5 × 10 ins. supplied water for the wet blow during sinking, and a steam pump 7½ × 5 × 6 ins. furnished water for cooling the air cylinders of the compressors and for mixing the concrete. A 5-HP. high-speed engine and generator operated a 110-volt circuit which furnished lights for the working chamber. A 9-HP. marine engine and a generator operated a circuit for the exterior lights. Quarters for the pressure men were provided in the stern of this barge.

The pile driver barge, 18 × 64 ft., was equipped with a hoisting engine 7¼ × 10 ins. and a 2,400-lb. drop hammer. This barge was used for driving the piles for launching ways, caisson moorings and fleet protection. The derrick barges were from 26 × 63 ft. to 42 × 83 ft., the depths being from 50 to 60 ins. For early construction one barge was equipped with a 66-ft. boom and 27-ft. mast, a second with a 54-ft. boom and 32-ft. mast, and a third with a 64-ft. boom and 40-ft. mast. Booms and masts were 12 × 12 ins., and the stiff-legs 10 × 10 ins. For later construction, the derrick of the second barge was raised on framed bents a distance of 30 ft. above the deck, and the third barge was equipped as a high derrick with a 66-ft. mast and 80-ft. boom, the boom seat being elevated 22 ft. above the deck. The mast and boom were 18 ins. diameter and the back stay and stiff legs were 10 × 10 ins., with 3 × 12-in. cross bracing. The boom seat

was supported on 12 × 12-in. posts and tied to the deck with 1¼-in. rods. The barge was 6 × 10 ins. high with a 3-in. deck, 6-in. walls and six 4-in. solid bulkheads. Engines with cylinders 7 × 12 ins. were used in operating the derricks. The concrete barge was 26 × 80 ft.

The cement house occupied an area 20 × 42 ft. and the remainder of the space was used for a concrete mixer, engine, and delivery platform. The cement house was 7 ft. high to the rafters, with a working capacity of 300 bbls. Material was taken in wheelbarrows to the mixer from material barges moored alongside. The concrete was dumped into a bucket set in a compartment in the hold of the barge and the bucket was raised by a derrick and dumped with a trip line. The material barges for sand and gravel were 30 × 80 ft., with bins 28 × 62 ft., 20 ins. deep, having a capacity of 135 cu. yds. A cement barge 26 × 64 ft. having a capacity of 1,125 bbls., was used to keep the concrete barges supplied. The material barges used for the transfer of cut stone were 24 × 60 ft. The signal lighter was a barge 12 × 38 ft., which marked submerged construction for the benefit of navigation.

The disposition of the fleet during caisson sinking was as follows: The power barge was moored on the downstream side of the caisson, the concrete derrick and mixer barge to the east, the material barges alongside, and the stone derrick and material barges to the west of the caisson and power barge. The fleet above described was used for the construction of Pier C and Piers 1 to 4. In the case of Pier D, a mixer was installed at the site of the pier and a stiff-leg 10-ton derrick was erected to handle the concrete. This derrick was equipped with a 30-ft. mast and 40-ft. boom. For Piers A, B, and E, the concrete was mixed on the concrete barge and delivered by derricks to tramways from which it was placed in the forms by means of stiff-leg derricks at the sites of the piers. The piling for these piers was driven by means of a drop hammer operated by the hoisting engine in leads suspended from the derrick boom.

The main substructure work comprised the handling of 1,000,000 ft. BM. of timber, 14,000 cu. yds. of excavated material, 7,000 cu. yds. of stone, and 15,000 cu. yds. of concrete. The timber includes that for permanent work, for the fleet and for temporary purposes. The substructure for the approaches required about 20,000 lin. ft. of concrete piling, 16,000 lin. ft. of wooden piling, and 4,000 cu. yds. of concrete. The concrete piling was driven by a hammer, as described above.

The concrete plant for the west approach had a concrete mixer mounted on wheels. The mixing platform was placed on the ground and the concrete was delivered to the forms by wheelbarrows running on inclined staging. A wheelbarrow elevator was used for high abutments. The concrete for the east approach was delivered in dump carts of 6 cu. ft. capacity operated on trestle work. This method was used for crossing a slough about 400 ft. wide.

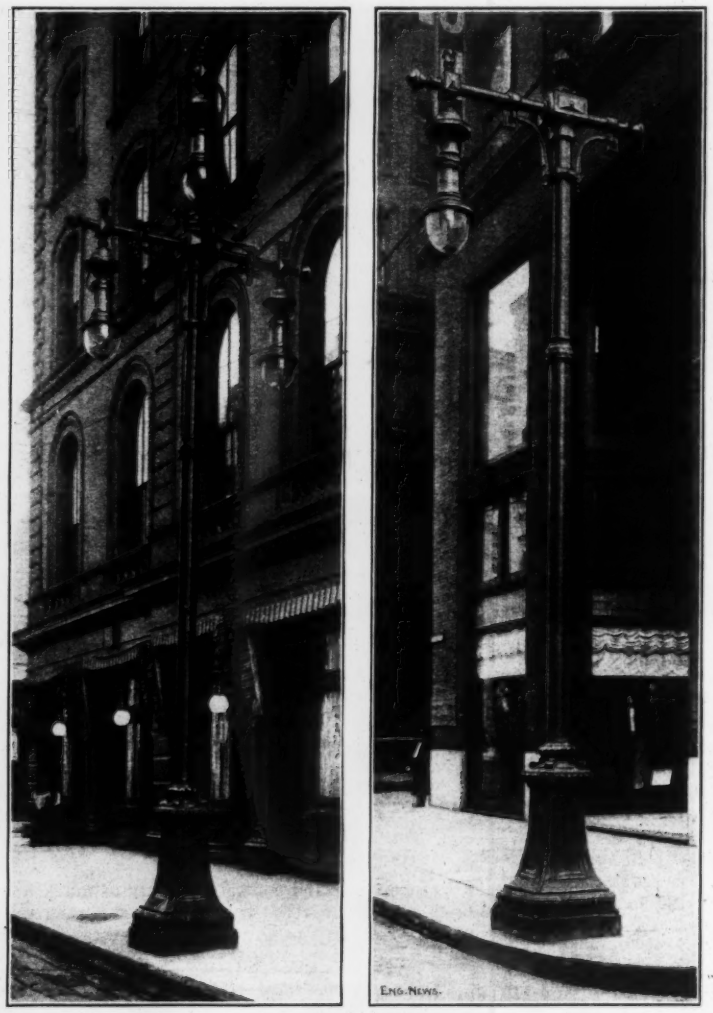
The wooden piles were used in a pile trestle which crossed a slough about 650 ft. wide. These piles were driven by a drop hammer which operated in leads pivoted at the top and held in position by an A-frame. The lower end of the leads swung on a moon beam to accommodate driving on a batter. The engine platform was made of sufficient length to counterbalance the overhang necessary to span one panel length. Caps were placed in position as the driver was moved forward and the bents were held in place by temporary struts.

Contractors.

The Missouri Valley Bridge & Iron Co., of Leavenworth, Kan., has the contract for the entire work on the bridge proper, including foundations, substructure and superstructure. Mr. E. H. Connor, M. Am. Soc. C. E., is Vice-President and Chief Engineer of the company, and Mr. J. B. McMahon is its Superintendent on the work. The American Concrete Co., of Chicago, had the contract for the approach substructure, and gave the Myers Construction Co., of St. Louis, the subcontract for driving the concrete piles and constructing the timber trestle.

Street Lighting in the Business District of St. Louis.*

It was long realized by merchants and property owners in St. Louis, that a new street-lighting system was badly needed in the business district and it was finally agreed, among the parties most affected, that definite steps for a change should be taken. The work of arousing sentiment in favor of a new lighting system in the business district and of carrying the project to successful completion was placed in the hands of the officers of a voluntary organization, composed of merchants and property owners within the business district, known as the Down Town Lighting Association. This organization began work early in 1909. Committees were appointed to make a thorough investigation of all existing forms of street illumination, and to ascertain what system was best adapted to the local conditions. After eight months it was decided to install the latest type of magnetite arc lamps. Therefore a committee was appointed to secure the design of suitable lamp standards, which could be made and erected at a moderate cost but which would be more dignified and ornamental than the unsightly old fixtures with their overhead service connections.



LAMP STANDARDS DESIGNED FOR THE BUSINESS STREETS OF ST. LOUIS.

These several matters having been settled, the plans were approved and adopted by the municipal Board of Public Improvements and by the Union Electric Light & Power Co. The city assumed liability for the bulk of the maintenance charges and the company undertook the installation of the new system, including the wiring and lamps, at its own expense, the only part paid for by private parties being the new standards.

It was estimated that the cost of the standards would be $30,000. To obtain this sum an assessment of $1 per front foot was made on all property fronting on the streets to be lighted by the new system. The response showed the movement to be popular.

The district thus illuminated is bounded by Fourth St. on the east, Twelfth St. on the west, Washington Av. on the north, and by Market St. on the south, comprising a total of 48 square blocks.

Formerly there were in this district only 138 arc lamps of the old enclosed type, which had been installed ten years previous. Under the plan of the Down Town Lighting Association there will be 437, 6.8-amp. magnetite lamps. These are placed in a uniform manner on all the streets in the district, with the exception of Broadway. This street was not included in the general lighting scheme, but was left to the City Improvement Association, composed of property owners and merchants on Broadway. This association installed on this single street 240 4-amp. magnetite lamps, the maintenance of which is paid for by this association.

The plan of the Down Town Lighting Association calls for four lamps at each street intersection, one on each corner and so arranged that two will line up with those on the north and south streets and two with those on the east and west streets. In addition, there are placed two lamps in the middle of the block, so arranged that they are approximately 80 ft. between centers. This gives a total of six lamps per block.

On Broadway the lamp standards are designed to carry three 4-amp. lamps; the standards are placed opposite each other on both sides of the street, and are at approximately 60-ft. centers. This gives a total of ten standards, or 30 arc lamps per block. This illumination is very brilliant, and has transformed the old dark Broadway into one of the most brilliantly lighted thoroughfares in America. When this work is completed there will be five miles of streets, brightly lighted by 667 lamps.

To furnish direct-current for the magnetite lamps the power company has installed three 75-lamp, 4-amp., and seven 75-lamp, 6.8-amp. mercury-arc rectifiers. It has also been necessary to increase the central-station capacity slightly.

The cost of the change, to the property owners in the district, exclusive of those on Broadway, was $30,000, and the installation on Broadway cost the City Improvement Association $10,000. The city having agreed to maintain all of the lights, excepting those on Broadway, the additional cost to it will be $30,000 per year, to be paid out of the general funds of the city. On Broadway, where the maintenance is borne by the merchants, the cost is approximately $9,000 per year. The cost of new equipment and of construction work assumed by the power company is estimated at $100,000.

The new system is permanent and will be in charge of Mr. Harry S. Sanderson, Supervisor of City Lighting.

*From information furnished by Mr. J. E. Tiedeman, Secretary of the Down Town Lighting Association, 124 North Fourth St., St. Louis.

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At the opening of the new year, Engineering News extends to each of its subscribers and advertising patrons its cordial greetings and good wishes for success and prosperity during the coming year. The year just closed has been on the whole a prosperous year for engineers, particularly in comparison with the "lean" year which preceded it; but 1910 promises even greater activity in engineering enterprises.

At the opening of a new volume, too, we wish to express our sincere thanks for the appreciative cooperation which we have so long had from correspondents, contributors and patrons, in many branches of the engineering profession and in many lands. It is our aim to make Engineering News of the greatest practical usefulness and interest to the whole engineering profession. The task assumes larger and larger proportions with the steady growth and development of the profession; but the opportunities increase in like proportion, and also, we believe, the necessity of the journal to the progressive engineer who aims to keep pace with developments in his profession.

We take this opportunity, also, to express our thanks for many expressions of cordial appreciation. Among the most gratifying features of the work which we are aiming to do for the engineering profession are the personal relations which exist between the editors and the circle of readers. It is a pleasure to work for the upbuilding and development of a profession whose foundation principles are honor, integrity and the truth, and whose success and advancement is closely related to the success and advancement of public morality, good government, efficiency in production and eventually the public welfare in its broadest aspects.

Mayor Wm. J. Gaynor, who became the official head of the New York City government on Jan. 1, began his term with some excellent appointments for the heads of Departments, a number of which are of especial interest to engineers.

Mr. Calvin Tomkins, who becomes the Commissioner of the Department of Docks, is equally well-known as a progressive business man and a worker for civic betterment in various important lines. Mr. Tomkins is a prominent figure in the brick, cement and crushed stone industry, is connected with several financial institutions, was President of the Municipal Art Society of New York in 1904-5 and has been for years a leader in the movement for intelligent city planning.

In connection with the Municipal Art Society he has given much attention to the planning of facilities for passenger and freight transit and has been a strong advocate of the policy of municipal control of its transit facilities. Mr. Tomkins is widely known among engineers and is an Associate of the American Society of Civil Engineers.

To head the Department of Charities, Mayor Gaynor has selected Mr. M. J. Drummond, the well-known cast-iron pipe manufacturer. Mr. Drummond is a Director of the Broadway Trust Co., the Emigrant Industrial Savings Bank and other financial institutions and is also connected with various manufacturing companies. He has long been interested in charitable work.

The new Commissioner of Gas, Water Supply and Electricity, Mr. Henry S. Thompson, is a man of large means, prominent in the building and real estate fields, was the organizer of the Thompson-Starrett Co., building contractors of New York City. He was in charge of the Department of Buildings in New York City under Mayor Low's administration and served for a short time after Mayor McClellan assumed office. He is connected with railways, insurance companies and banks, and should give the city a business-like administration of this important Department. For First Deputy of this Department Mayor Gaynor has appointed Prof. Edward W. Bemis, of Cleveland, whose record as head of the Cleveland water supply department under Mayor Johnson and whose work in the introduction of the meter system in Cleveland are well known. This going outside of New York City to secure a man with reputation and experience gained elsewhere, is a most gratifying and encouraging departure from usual customs in municipal appointments. It is currently reported that Prof. Bemis is to be especially charged with the work of reducing water waste in New York City.

As Commissioner of Bridges the Mayor has appointed Mr. Kingsley L. Martin, M. Am. Soc. C. E., who has been Chief Engineer of the Bridge Department since the retirement of Mr. C. M. Ingersoll. Mr. Martin's appointment is in the line of continued promotion. He is a son of the late C. C. Martin, who was the Chief Engineer of the Brooklyn Bridge from its completion until his death. Mr. Kingsley Martin entered the Bridge Department after graduating from Stevens Institute and has been steadily promoted until he has attained his present position. It is very gratifying to the engineering profession to have the Commissionership of the Bridge Department again held by an engineer, a condition which has not existed since Mr. Lindenthal's term of office in Mayor Low's administration. While executive ability is even more important than technical attainments in the Commissioner's work, the advantages of a thorough acquaintance with engineering are very great, and it is to be hoped that Mr. Martin will place his Department on the highest level of efficiency it has ever reached.

It is well worth noting in this place that the new President of the Borough of Manhattan, Mr. Geo. McAneny, has appointed as head of the Bureau of Buildings, Mr. Rudolph P. Miller, M. Am. Soc. C. E. Mr. Miller's previous experience in the Department of Buildings and his technical qualifications make this appointment an ideal one.

We have deemed these appointments worthy of special notice in this prominent place, since New York's position makes her successes or failures in municipal administration of interest the country over.

A large and interesting book might be written on the "Vicissitudes of Engineering Projects." The idea is suggested by the interesting article on "The Proposed Lake Washington Canal at Seattle, Wash.," etc., published elsewhere in this issue.

The Lake Washington Canal project seemed so well assured of realization years ago that from 1894 to 1897 we published several articles on its main features, as then outlined. During the fifteen years that have passed since the earlier of the two dates Seattle has grown marvelously and the reasons for building the canal have multiplied in number and weight, but the project is still a project.

Mr. Powell's article describes the eugineering and commercial features of the proposed canal and outlines the reasons for the long delay in making it an accomplished fact. These reasons have been and are common to many other worthy engineering projects. They comprise various difficulties incident to bringing many conflicting interests into harmony. Overcoming such obstacles may seem to be no part of the task of the engineer, but Mr. Powell's article shows the service that an engineer may render the public by a clear-cut forceful statement of the nature and merits of an engineering project and the obstacles in the way of its fulfilment. To bring such facts as he narrates before the public should aid materially in making a great engineering project a reality. Mr. Powell's article seems likely to be an effective aid to that desirable end in the case of the Lake Washington Canal.

We have been favored by a correspondent with a copy of a prospectus entitled "Electricity from Sunlight." It describes a scheme for developing electricity from the direct heat of the sun which has been invented by George H. Cove, of Somerville, Mass., and has been more or less exploited in some of the newspapers and popular journals during the last few months.

According to Mr. Cove's claims, he has built a battery of thermopiles 4 ft. square which, when exposed to the direct rays of the sun, generates daily sufficient electricity to light six 16-c.p., 10-volt tungsten lamps for four hours. At this rate a rough computation indicates that something like 300 sq. ft. of exposed surface would be necessary to develop a horse-power during the hours when the sun shines. What would drive the "sun electric power station" during days or weeks of cloudy weather when the sun does not shine, the prospectus does not state. It was clearly shown in the article which we published in our issue of May 13, 1909, describing the work of two other inventors in the field of sun power, that, without large provision for cheap power storage, there is hardly a commercial possibility in sun power development.

Mr. Cove, it is true, uses an electric storage battery with his experimental machine; but power is so cheaply developed nowadays from steam and gas engines and water power that the storage battery cannot be economically used except to carry "the peak of the load."

According to the prospectus, the inventor's work so far has been confined to the making of one experimental thermopile, four feet square, cost and materials used not stated. The inventor makes affidavit that he "will be able to generate enough electricity through Sun Electric Generators to meet any demand for light, heat and power for commercial purposes"—cost also not stated.

On this basis the prospectus figures out profits in this enticing fashion:

> Suppose, for the sake of illustration, we say the company can make 100,000 generators every year of one particular style, say for lighting residences, and say the price is $100 each, with a yearly rental as royalty of $20 each; then the estimated total amount received would be $10,000,000 each year for installation charges, besides royalty receipts increasing at the rate of $2,000,000 a year.
>
> Allowing for cost to manufacture and a liberal margin for operating expenses, the expenditure for manufacturing and selling should not exceed an estimated figure of $5,000,000.
>
> This would give an estimated net profit of $5,000,000 on the manufacture of 100,000 generators annually. The total capital of the Sun Electric Generator Co. is $5,000,000. A profit of $5,000,000 would pay dividends of 100%. But in addition to this would be the rental or royalty receipts, amounting to $2,000,000 the first year, $4,000,000 the second year and so on.

The prospectus continues in a similar vein and rolls up prospective earnings to still dizzier heights.

We are strongly reminded of Æsop's story of the milk-maid,' who computed the profits from the chickens to be hatched from the eggs to be purchased by the sale of the milk in the pail she was carrying home balanced upon the top of her head. From these profits she would purchase a gown which would secure her a lover; but trying some steps of the dance for her wedding, the pail of milk fell to the ground.

To come down to more recent times, we are filing this prospectus with a bundle of similar documents which were circulated a dozen years ago to show investors how to get rich quick by investments in liquid air power schemes.

Young engineers out of employment or with an eye to preferment in fields out of the ordinary might find it to their advantage to look into the present shortage of enumerators for the next Census of the United States. At the joint meeting of the American Statistical Association and the American Sociological Society held in New York City last week, Mr. E. Dana Durand, Director U. S. Census, stated that the number of persons who came up for examination last November for possible appointment as field enumerators on the manufacturing schedules was so small that it might be necessary to appoint all who passed the examination instead of confining the appointments to those who made a good showing. Perhaps it is too late now to get on this particular list of "availables," but future possibilities of the same general kind are worth keeping in mind. The work on the Census manufacturing schedules is of a sort that would certainly not be uninteresting to most bright young engineers, and the increasing demand for technically-trained men in the Bureau of the Census and in other government departments suggests a future for those who gain experience and show ability in Census work.

A less promising but far wider demand for Census enumerators has just been given publicity by a circular issued by Mr. Durand calling attention to the difficulty of getting general enumerators to undertake the population, agricultural and minor manufacturing schedules of the Thirteenth Census. Field work on these schedules will begin April 15, 1910. The period of employment will be short and the compensation not large, but it is quite easy to imagine circumstances under which a few, at least, of the readers of this journal, or many sons of readers, might take up the work. Altogether, thousands of enumerators are to be employed throughout the length and breadth of the land. Two months spent on our great national Census would be a valuable experience for anyone who could give the time to it without undue sacrifice, and it might open up a useful life work to a young man.

It should be remembered that the Thirteenth Census, compared with previous ones, will be removed from the field of politics; also that we now have a permanent Census Bureau, which, between each decennial Census, carries on a number of interesting special investigations. There is thus a continuing demand for men who can show experience and capacity for statistical work.

In conclusion, we may note as an encouragement to present and prospective technically-trained men of all classes that one of the dominant notes in the discussions before the economic, statistical and other conventions held in New York City last week was the increasing demand for trained men in the government service—not only men trained to observe, disclose and record facts, as in a national Census, but also men of technical attainments which qualify for various administrative positions in the national, the state and the municipal service. The engineer has already been given considerable recognition by those seeking trained men for government positions, but unless he is ever on the alert to broaden and strengthen his attainments in economics, in statistics, and in politics of the better sort he will fall behind in the race that is now barely opening.

Reducing the Rust Loss.

The production of low-impurity ingot iron—equivalent to a pure wrought-iron, but produced by the large-scale process of the steel furnace—must be ranked as one of the most important innovations in the modern metallurgy of iron. It promises to cut down the world's prodigious rate of rust loss to a fraction of its present amount, and in this fact lies its importance. The small disadvantages affecting the process are negligible when set over against the gain. We may never hope to obtain a "noble" iron, one which will not rust at all; but just in degree as we ennoble the perishable wrought metal of the day by reducing the rapidity of rusting, we raise it in rank and in value.

Lower strength and higher cost, as compared with our present standard mild or low-carbon steel, are the disadvantages of the new iron, whose manufacture is described on another page of this issue. Lower strength means that generally more metal must be used to obtain the same initial strength of structure, though many uses would not be affected by the lower strength (as, for example, fence wire or mesh would not need to be heavier than if made of steel). In any case the extra metal may also be expressed as cost: we must pay more for a structure composed of the new metal not only because the metal costs more to make per pound, but also because more pounds must be used to get the same strength.

But is this item of cost really a serious matter if we gain largely increased permanence? Suppose the new metal endures but twice as long as the kind it displaces. Will a cost-increase of 25 or 50 or even 100% wipe out the economy given by the doubled life?

The field which an iron of high rust-resistance is destined to command is exceedingly large. Primarily it will include all exposed bare iron, that is, all iron which is or must be used outdoors under such conditions as make it impracticable to protect by paint or other coatings. This in itself is a large field. But the material would seem hardly less valuable for the outdoor uses of protected metal, that is, where at present we rely on a coating of tin or of zinc or of paint. Thus, evidently, the field open to rust-resisting iron comprises by far the largest part of the uses of wrought metal. Consider only bridges and ships!

The lack of efficiency of tin and zinc coatings has been the subject of so much serious complaint in recent years that one might almost believe such coatings to be entirely worthless. Tinned roofing sheets have been reported as rusting out with surprising rapidity, certainly many times more rapidly than roofing sheets made thirty or forty years ago. Such a product as galvanized fence wire seems to show a startling rate of corrosion, and many people believe galvanized wire of the present day to be no better than plain wire of former decades, if it be as good.

The question, of course, is complicated by the coating, in the case of both tin and galvanized material, and matters like thickness, purity and uniformity of coating have such intimate relation to the durability of the whole as to make it difficult to reach definite conclusions concerning the underlying metal itself. The scientific study of the matter has been begun in a promising way, and may in time lead to a precise analysis of the factors involved. Up to the present, the only definite thing we can say is that quite probably the high impurities in modern soft steel do affect the permanence of the plate even when tinned or galvanized, and do make such articles of shorter life than similar articles of a purer metal, as wrought-iron.

If this view, which is popular and rather plausible, should prove to be supported by fact in the long run, it would mean that a metal like the new ingot iron would be bound to force its way into the tinning and galvanizing industry. If it did not wholly displace steel, it would at least crowd it back into a subordinate place in that field. The difference in cost might prove a temporary hindrance to its introduction, but could not prevent it for long. If the purer metal be economical in those situations where it cannot be protected by a covering or coating, then any marked increase of life of coated sheets by its use must also be advantageous; for the coated metal, being costlier than plain metal, would be relatively less affected in price by increased cost of a single one of its parts, namely the iron.

The great field of painted steel, however, is probably the most important for the application of rust-resistant iron, strange as it may seem. The use of paint as a protection against rust really only aggravates the urgency of the rust question. The truth is, paint protects but a very short time. Painting troubles are ever present and ever vexatious. And if anything can be produced that will place less responsibility on the paint, and, by making the paint covering less important, render it less of a worry, it will prove an enormous boon to the engineer.

Our use of iron has grown so wide that the paint question has in recent years taken on a remarkable prominence. This, in turn, has led to active study of paints during the last decade or more, in both practical and scientific directions. But despite all the theorizing and experimenting that has been put on record, paints have experienced practically no improvement. The old-style paints still hold their place, and we still are compelled to place chief reliance on linseed oil. At the same time, rust troubles have become if anything more pronounced with years. The greater extent of use, and the increasing severity of exposure, of iron structures—probably also the greater non-uniformity of the metal composing them—have all increased the evil of rusting.

The world seems to be ready for a radically new departure in metal painting—something that will free us from the limitations of linseed oil as now used and will give us a lasting efficient protection. There is no promise of such a development, however. To all appearances, we will be dependent on linseed oil and bound by its weaknesses for many years to come. Improvement seems possible in only two directions—perfecting the technic of painting in order to obtain a coating of greater strength and solidity and longer life, or using a less perishable metal beneath so as to be less dependent on the protecting power of the paint. The former resource has been somewhat explored, and there is no present prospect of improvement in that direction. The other, however, is now concretely placed before us in the shape of ingot iron.

Let it not be assumed, however, that purified iron will enable us to dispense with painting, in any perceptible degree. Paint for iron will always be with us. Iron is essentially a rapidly rusting substance, and it will always need a covering just as man needs clothing. Improvement in iron will never threaten the prosperity of the paint industry. But it may, and we hope will, relieve paint and the paint industry of a good share of the excessive importance and responsibility which has been forced on them.

We said above that the new metal *promises* to reduce the rust loss. It has not yet *proven* its ability to do so, in our opinion. There are cited various corrosion records in sulphuric acid solutions, of surprising character, showing for instance that the weight loss of the new iron is only a few per cent of the loss which structural steel experiences under equal exposure. If the same kind of test, under a variety of conditions, with independent authorities in charge, will substantiate these records, an excellent prima facie case for the claims of the "ingot iron" will be made out. The highest expert opinion has established the sulphuric acid test as our best available accelerated test of rust-resistance. Yet, after all, performance under actual atmospheric exposure must give the final comparison. The test of time alone will prove the case.

In the interim, however, the promise of less corrosion is sufficient to warrant the most serious consideration of the new metal for all service affected by corrosion. The probability of a large gain is too attractive to be passed by. So soon as the metal becomes commercially available in forms other than sheets it should enter

into most varied use. Water-tanks and towers, buried pipe, ship-plating, short-span bridges—these and dozens of other structures or elements will at once put in a claim for its use.

LETTERS TO THE EDITOR.

Method of Drawing Casing in Concrete Piles with Enlarged Bases.

Sir: In answer to certain inquiries regarding the method of drawing the casing used for the shell in the concrete piles described in my article "Concrete Piles With Enlarged Bases," (Eng. News, Dec. 16, 1909, p. 685), I beg to submit the following:

A pulling frame is built up inside the pile driver frame with its front timbers bolted to the rear of the leads. This pulling frame is about one-half to three-quarters of the height of the pile driver proper. A channel cross beam rests on top of the frame and a ten-fold pulling tackle is hung from it, and a 1¼-in. cable sling is attached to the hook on the lower block and loops around a steel collar on the outside of the upper end of the casing. This pulling tackle is operated from the second drum of the hoisting engine and exerts an enormous pulling force, which is capable of withdrawing the casing under the most adverse conditions.

Yours very truly,

Hunley Abbott.

11 Pine St., New York City, Dec. 30, 1909.

Asphalt Specifications.

Sir: I have read with great interest the letter in your issue of Nov. 11, 1909, written by Mr. H. W. Clark, Chemist of the Massachusetts State Board of Health [and Massachusetts Highway Commission—Ed.] in answer to an article of mine on "Confusion of Asphaltic Specifications," printed in your issue of Oct. 7, 1909. As I purposely asked for criticism I am indeed glad that an interest has been shown in this matter. There are certain statements advanced by Mr. Clark which in justice to myself and the company with which I am associated (The Texas Co.) I desire to correct.

It may not be amiss to state here that the former article discussed in an entirely impersonal manner, some controversial points regarding asphalt specifications, which matters have been similarly discussed in the same impersonal manner by the writer in previous articles published in various engineering journals. These criticisms were the result of my own practical experience in road building and paving and the experience of others.

In the first place, in Mr. Clark's article he states that it is evident "that these specifications (the specifications referred to in my previous article) are made to fit the favorite product of that company" (The Texas Co.). I would point out that this is an entirely erroneous position inasmuch as it is just as easy for this company to produce and sell residual oils of a semi-asphaltic character similar to those which are mentioned in the specifications criticized and that such oils could in fact be produced with greater ease.

The specifications referred to in my previous article were advanced as the basis of a standard by Dr. Albert Sommer, at the last annual meeting of the American Society for Testing Materials, and were advanced entirely upon their merits as a scientific and practical proposition. In fact, it is evident that in submitting these specifications to the consideration of so eminent a body of engineers, it would be foolish to do so upon any other than properly practical and scientific grounds.

In further confirmation of the correctness of the attitude taken by Dr. Sommer, I would point out the paper read by Mr. Prevost Hubbard, Assistant Chemist of the U. S. Department of Public Roads, before the Good Roads convention at Cleveland in September, in which he expresses the following principles as applied to macadam binder:

(1) The elimination of all heavy oils.

(2) Liquefying of the asphaltic compound only by the introduction of light distillates.

(3) The necessity for demanding rather than excluding a certain percentage of volatile products.

With such corroboration of the practical value of the specifications which I submitted in your issue of Oct. 7, it is unnecessary to criticize further the particular tests to which Mr. Clark refers or to discuss further the scientific value of the specifications advanced.

I wish, however, to state that The Texas Co. is in a position to supply the product described in Mr. Clark's specifications, and the only reason it has not, and does not intend to supply such oils for macadam road work is that it is the opinion of its chemist and engineers that such oils are not suited for the work; and entirely from commercial considerations, The Texas Co. desires and expects to put on the market only those materials which (as a result of its investigation) are deemed of most practical value to the engineer.

I trust that Mr. Clark will absolve me from any intention of personal criticism of any kind and see that the matter was put forward entirely as in my opinion a progressive step towards the arrival at a solution of the best practical macadam binder for road building.

Yours very truly, H. Tipper,
Publicity Engineer, The Texas Co.

17 Battery Place, New York City, Dec. 8, 1909.

Concrete Pavements.

Sir: Engineering News of Dec 23, 1909, prints the paper of Mr. Logan Waller Page on "The Possibilities of Portland Cement as a Road Material" and your editorial comments thereon. Both writers speak disparagingly of plain hydraulic cement concrete pavements, and it is evident that both are inclined to regard them as hopeless failures. The writer does not think that this conclusion is warranted by the facts. To be valid, such a conclusion must be based on sound theoretical reasons or on adequate experience. Neither theory nor experience has yet demonstrated that these pavements may not prove to be successful within the limits of their appropriate field of usefulness.

It has never been claimed by disinterested persons who are qualified to express an opinion upon the subject, that a hydraulic concrete pavement is suitable for use on streets other than those carrying a comparatively light travel, such as suburban roads and secondary or residence city streets. The extravagant claims for great durability under heavy travel, made by the promoters or patentees of specially constructed concrete pavements, may safely be disregarded. All that can be fairly claimed for it is, that for streets or roads of moderate travel, it will prove much more durable and satisfactory than macadam and will cost much less than the standard high-cost pavements. In other words: its appropriate field is in the wide gap that now exists between macadam and such costly pavements as asphalt, brick and wood block. Theoretically, the pavement is well adapted for use in its proper sphere. It is true that concrete is somewhat brittle, and disintegrates along exposed edges under travel, but this is true also of asphalt and bituminous macadam, and the edges of wooden blocks are soon battered down where there is no lateral support. It is said that concrete pavement is hard, and breeds lameness in horses; but it is not more objectionable in this respect than stone-block and brick, and asphalt pavements are equally hard and rigid during about one-half the year. If the concrete be made only thick enough to endure the travel, and laid on earth sub-foundation, it will have sufficient "spring" under the impact of the horses feet to mitigate the anvil-like quality we usually associate with concrete. A concrete roadway presents a smooth surface, and yet affords superior foot-hold for horses.

To the question of durability, only longer experience can give a satisfactory answer. The weight of present evidence is to the effect that, where properly constructed, concrete pavements possess a high degree of endurance under the weight of the travel for which they are appropriate.

It must be remembered that all new kinds of pavements require time and trial to develop the proper composition and the best method of construction, and that in not a few of the comparatively small number of concrete pavements so far built, the composition and manipulation of the concrete was, to say the least, faulty. The fact that a very respectable number of them are reported to be giving satisfactory service proves that this kind of pavement merits further experimental study. Why condemn them before they have had a fair trial?

While engineers will await with interest the result of further experiments by the national Office of Public Roads looking to the development of a combination of oil and hydraulic cement, it may be pretty safely predicted that salvation does not lie in that direction. It will probably be found that oil or bitumen, when mixed with the green mortar in sufficient quantity or ratio to be of value in making the road surface elastic and dustless, will prevent the setting of the cement, or so far reduce the strength and hardness of the resulting compound that any other fine, inert mineral matter might be substituted for the cement with almost equally good results.

In the opinion of the writer, a more promising field for experimental study would be in the direction of some practicable method of saturating the mortar with petroleum oils after the concrete has become fully set.

S. Whinery.

New York, Dec. 28, 1909.

[The editorial referred to admitted the "successes of . . . this construction on lightly traveled streets," which would seem to agree with our correspondent's phrase—"successful within the limits of their appropriate field of usefulness." On heavily traveled streets the burden of proof is still on the advocate of the concrete pavement.—Ed.]

Concerning High Vertical Web-Compression in I-Beam Tests.

Sir: Your issue of Dec. 9, 1909, contains a communication from Mr. C. W. Hudson relating to the tests of steel I-beams reported by the writer at the last annual meeting of the American Society for Testing Materials, and published in abstract in your issue of Aug. 12, 1909. Mr. Hudson states that in his opinion "the results of these tests should not be taken to represent the action of such beams under the loadings of practice." In support of that position he says, in substance, (1) that in practice the loads on beams are usually uniform, or approximately so, and (2) that the element of direct vertical compression in the vicinity of concentrated loads, especially in the absence of stiffeners, has an important influence on the maximum intensity of stress in the web.

As to the distribution of the loading, it may reasonably be held that deep I-beams, 24 and 30 ins. in depth, are, in practice, more commonly subjected to locally concentrated than uniformly distributed loads. However that may be, the former is obviously a very common condition.

Most of the tests in question were made under quarter-point loading. Nine of the 15-in. beams were tested also under central loads. The comparison, instead of proving disadvantageous to the latter, as might be inferred from Mr. Hudson's discussion, was, on the contrary, greatly in their favor, the elastic limit in bending being 45.0 and 19% and the modulus of rupture 22.22 and 31% higher for the Bethlehem Special beams, and the standard beams and the Bethlehem Girder beams, respectively, each value representing the average of three tests, the beams of each section having been rolled from the same heat of steel. The writer confesses that he is unable to suggest a satisfactory reason to account for these observed facts. As stated in the paper, a partial explanation may, perhaps, be found in the circumstance that with the former (quarter-point) loading, the maximum fiber stress is developed throughout the middle half of the beam, instead of at the central section only; although this applies with equal force to the standard beams which exhibit no such difference in their behavior, in so far, at least, as the elastic limit is concerned.

That the metal in the upper part of the webs, under and near the 12-in. bearing plates through which the loads were applied to the upper flanges, was subjected to considerable vertical compression is, of course, true; and, as Mr. Hudson rightly points out, the lines of maximum stress intensity in the web will, in consequence, become nearly vertical in that region. In the original paper attention was directed to this feature by the following reference to a photographic reproduction of the lines of principal stress in the web, as revealed by the flaking of the mill-scale, and their sharp definition by rusting on the subsequent exposure of the beam in the open air:

> The lines of principal stress along the upper flange show the influence of the local concentration of the load. Here the directions of the principal lines are nearly vertical, sloping slightly away to either side from the point of application of the load.

In considering the combined effect of the vertical and horizontal stresses acting on a horizontal parallelopiped in the web above the neutral axis, it is to be observed that each set of stresses induces tensile strains (elongations) in planes normal to the direction of these stresses, and therefore strains opposite in character to the compressive strains from the other set of stresses at right angles to the first. If, for example, a cube be subjected to compression by forces on two opposite faces, the resulting stresses are greater than if the cube be subjected also to compressive forces of like intensity on two other opposite faces.

It is true that the presence of stiffeners riveted to the web and tightly fitted against the upper flange would affect the distribution of the vertical compressive stresses in the web, although in a quantitative sense the resulting distribution would be quite indeterminate. It is not believed, however, that the principal compressive stresses in the webs when stiffeners are absent deserve to be regarded as a primary source of weakness of the beams for the conditions under which they were tested, or that the presence of such stiffeners would have materially affected either the deflection or the fiber-deformation curves on which all the more important deductions in connection with the bending tests are based. At least, the writer is unable to discover any valid ground for such inferences. As stated in the paper, the maximum shears developed in the Bethlehem Special beams and Girder beams are, with a single exception, slightly less than twice as great as the maximum safe shears according to the formula given in the Bethlehem Steel Co. hand-book, and the values from the latter are stated to have a safety factor of from three to four with respect to the shear which would produce crippling. In the case of the single exception referred to, namely the 24-in. Special beams, the maximum shear developed in the tests was only 6% greater than twice the maximum safe shear by the formula. In no instance was there any evidence of the buckling of the web in

detail vertically, not even at the points of loading and support.

The features of these tests which deserve to be especially emphasized are (1) that the material was shown to possess elastic limits as low as 11,000, 15,200, 16,300 and 17,100 lbs. in pure tension, as observed in the tests of specimens cut from the central or thickest part of the flange; and (2) that the elastic limit, as well as the modulus of rupture, decreased very rapidly with increasing depth of beam, the former reaching a value as low as 10,800 lbs. for one of the three 30-in. Bethlehem 175-lb. girder beams, the average value for the three beams of this series being only 11,800 lbs. These values of the elastic limit are the computed extreme fiber stresses corresponding to the loads above which the load-deflection ratio ceases to remain constant. Corroborative evidence of the true significance of these values is afforded by the fiber-deformation measurements made on a gage-length of 40 ins. by extensometers reading to 0.0001-in., for which the contact points on the flanges were symmetrical with the centers of the beams subjected to quarter-point loading, the fiber stress being constant between the points at which the loads were applied. For the beam above referred to which exhibits the lowest elastic limit in bending, viz.: 10,800 lbs., the average elastic limit based on direct fiber deformations of the upper and lower flanges has the value 11,100 lbs., or practically the same as the value in bending. The readings of the extensometers on opposite sides of both the upper and lower flanges were substantially in accordance for the entire range of observations, which extended far beyond the elastic limit, thus showing conclusively the absence of lateral deflections in the flanges.

The question raised by Mr. Hudson is one of many to which a conclusive answer can come only from actual tests, although in the judgment of the writer, it is of secondary importance in comparison with other questions relating to the behavior of steel I-beams awaiting further experimental inquiry, which, it is to be hoped, will be undertaken at no distant date.

Edgar Marburg.

University of Pennsylvania, Philadelphia, Pa., Dec. 13, 1909.

Bending Between Web and Flange in I-Beams Illustrated by a Model Test.

Sir: The writer recently made a test of two model I-beams formed from tin-plate, whose results teach a lesson in bracing. The purpose of the tests was to determine the value of sidewise support of the top flange of a beam.

Two beams were made as near identical in every way as possible, with one small exception. They were made of one sheet of roofing tin. This was cut into four strips and each was bent by a tinner into the shape of a channel. The depth of the channels was 3¾ ins. and the width of flange was ⅝-in. They were 28 ins. long. The channels were riveted together back to back in pairs, to make two I-beams. Little strips of tin were bent over the bottom flanges at the supports and over the top flanges at the middle of the beam, where a load was to be hung, to prevent spreading. In all of these respects the beams were alike. One of them was now modified as follows: The metal was slit at the heel of channels for ½-in. back from each end, and the web was bent out. The little piece of flange was bent down and a rivet driven into it, uniting it with the web to form an end stiffener. In this manner one beam was made better than the other by the sum of eight little copper rivets and sundry bends in the web and flanges at the ends of the beam.

The beam without the end stiffeners was placed so as to span a clear space of 25½ ins. from one desk to another. A stirrup composed of a block of wood and a heavy wire was hung at the middle, with the block acting as a bearing on the beam. A tin bucket was hung on this wire, and bolts were placed in the bucket. The surprising load of 83 lbs. was thus hung on this frail beam, and it keeled over to the right at one end and to the left at the other, with a large twist in the web, dropping the load. The top flange remained almost straight. The bottom flange was almost straight laterally but had in it a large vertical curve. The writer has had material straightened up and put into a structure that was in no worse condition than the flanges of this beam after ultimate failure of the beam.

The other beam, the one with end stiffeners, was likewise placed on the supporting desks, and the bucket was hung upon it. By the help of a board across the top of the bucket a weight of 190 lbs. was hung on the beam. The ends remained plumb, but the top flange failed by a sharp buckle close to the applied load.

The two pictures herewith show the appearance of the tin beams after failure. The "twisting of web" in the first beam is clearly apparent. The weight added to Beam 2 to make the additional strength of 129% was a small fraction of one per cent.

These tests were undertaken to investigate the value of bracing. The end stiffeners of the girder were not for the ordinary purpose of stiffening the web against buckling but for the purpose of preventing the top flange from swaying laterally. The unstiffened beam did not buckle in the web at the end, but it did keel over. By preventing the leaning of the web at the ends of span these stiffeners added to the ultimate strength of the beam by a percentage very much larger than that by which the ultimate strength of the metal exceeds its elastic limit. The bearing and significance of this last statement will appear.

Some months ago Prof. Edgar Marburg described some tests on beams before the American Society for Testing Materials. Some of these showed exceedingly

low elastic limits and ultimate strength. They were not braced laterally, and when this fact was pointed out on the floor of the convention, it was stated by Prof. Marburg and Prof. A. N. Talbot that the lateral support is of no influence until the elastic limit is well past. The worst of these beams were high ones with limber webs. The failures of these high beams were exactly like the little model beam above described, which was not steadied at the ends by the stiffeners. They failed by "twist of web." In the pamphlet describing the tests some photographs are shown of beams after failure. One in particular, Fig. 23, shows a very large difference in inclination of the web at the two ends. This difference is about 26°. The elastic limit of this beam was low, being only 13,000 lbs. per sq. in.

This elastic limit was gaged by deflection and calculated from the theory of flexure. Now a lean of 13° at each end of a 30-in. beam with the web vertical at the middle, it can be readily shown, would in itself give a deflection of ⅜-in., as this is the drop that the middle line of the beam would take due to this leaning of the web. The beam in question had a deflection just before failure of ½-in. When nearly all of the deflection of the beam can be accounted for by this lean of the web, and when this lean could have been prevented by holding the top flange laterally, and when a beam in practice would invariably be so held, is it proper to blame the low strength and low elastic limit on "very inferior grade of material?"

In the writer's tests of the unstiffened beam the leaning at the ends of the web began long before it failed. This leaning would contribute to the deflection in very large amount and would make the elastic limit appear to be very much less than it really is.

A beam flange is not designed to resist the stresses that would exist in this twisted beam, and the web is not designed to resist horizontal deflection of the flange at its ends. The stresses calculated for the flanges by the theory of flexure are not realized in such a contorted beam, and hence deductions based on these calculated stresses are of no value whatever.

There is no doubt whatever that the high beams tested by Prof. Marburg failed under such low loads because the webs leaned sidewise and were not capable of holding the ends of the flanges. The lower beams showed greater strength because the webs were not so limber. This same kind of test could be made on a plate girder and its elastic limit could be found as low as desired, if such ill considered tests be taken at their face value.

In a pair of cardboard beams tested by the writer one showed almost double the strength of the other merely because the top flange was held against sidewise movement by books laid against it at the supports.

The flanges of the 30-in. beam tested by Prof. Marburg could have been much stiffer laterally without preventing the twisting. This explains why the high beam with the wide flange showed up so much weaker than the low beam with the more slender flange. It explains how a permanent set could be observed in a beam when the elastic limit of the flange had not been reached.

Prof. Marburg's tests prove conclusively that lateral support or the lack of it has a whole lot of influence long before the elastic limit is reached, in spite of authoritative statements to the contrary. In the writer's tests the ultimate strength of the unsupported beam was much less than the elastic limit of the other can possibly have been, being less than 44% of the ultimate strength of the supported beam.

One editorial writer because of Prof. Marburg's tests and his conclusions therefrom has been led to suggest the advisability of recasting current methods of designing beams. A little more digestion would prove that these tests and conclusions are not fit to be assimilated into engineering practice. They violate the fundamental principles of engineering practice because of the unsupported condition in which the compression parts were left. The same editorial writer berates the destructive critic for trying to tear down the fabric that these tests and conclusions have built up. If destructive criticism can tear down error it has the highest kind of constructive influence. Plain reasoning without tests is enough to "controvert the conclusions to which those tests seem to point." There need be no apprehension felt on the part of engineers "that the unit stresses in current use are excessive," unless the tests that show it are arranged as beams in current practice would be.

Edward Godfrey.

Monongahela Bank Bldg., Pittsburg, Pa., Dec. 4, 1909.

Notes and Queries.

G. H. L., in charge of a large public library, is having difficulty with the floors, and seeks advice from the experience of others. The building has bare concrete floors; some of them must be covered to give a soft and noiseless walking surface, and here cork matting has been adopted. The architect knew that this covering was to be used, but did not provide means for fastening it down, and now the cork matting must be glued down. The adhesives tried so far have proved unsatisfactory; they do not stick to the concrete, and especially at the joints the matting shows cracks. There is also the question of treating the surface of the cork matting with an oil or the like, to make the floors easier to clean and to preserve the cork. What oil or other substance is suitable for this purpose? In other parts of the building the concrete is left bare, but it is too hard to clean and therefore should be oiled or painted. Information about good materials for this use is desired.

A Rational Method of Determining the Principal Dimensions of Water-Turbine Runners.

By S. J. ZOWSKI.*

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The principal dimensions of a water turbine runner are determined from the required speed and capacity and the available head of water.

Let D = the mean runner diameter, in feet.

v = the corresponding peripheral speed, in feet per second.

N = the required rotative speed, in revolutions per minute.

then $$v = \frac{\pi DN}{60} \text{ and } D = \frac{60v}{\pi N} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

so that when the proper peripheral speed to give good hydraulic performance is known, the diameter of the runner follows therefrom.

Let us assume that the runner is designed in such a way that at its best speed the water discharges from the runner buckets in planes going through the axis of rotation; this is a condition which the turbine designer should always attempt to secure, in order to avoid helical stream lines in the draft-tube. Then the best peripheral

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speed is given by a simple formula. Denoting the bucket angle by β, the guide-vane angle by α, as in Fig. 1, and the hydraulic efficiency by e_h, the formula is:

$$v = \sqrt{e_h\, g\, H} \times \sqrt{\frac{\sin(\beta - \alpha)}{\sin\beta \cos\alpha}} = K_v \times \sqrt{H} \qquad (2)$$

where

$$K_v = \sqrt{e_h\, g} \times \sqrt{\frac{\sin(\beta - \alpha)}{\sin\beta\cos\alpha}} = \text{speed constant} \qquad (3)$$

The curves in Fig. 2 give the values of the second radical for several constant values of bucket angle β with varying values of guide-vane angle α. The following limits for α and β appear reasonable.

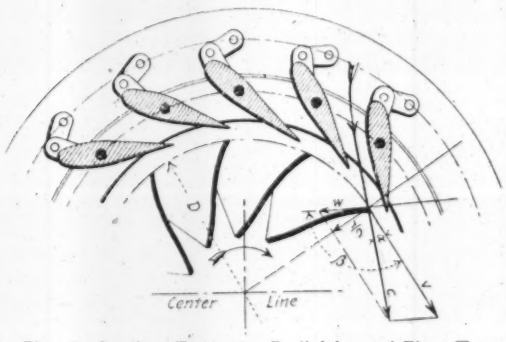


Fig. 1. Section Through Radial Inward-Flow Turbine, Showing Relation Between Bucket and Guide-Vane Angles.

For a pronounced low-speed turbine $\beta = 60°$, $\alpha = 20°$; then

$$\sqrt{\frac{\sin(\beta - \alpha)}{\sin\beta\cos\alpha}} = 0.88$$

For a pronounced high-speed turbine $\beta = 135°$, $\alpha = 40°$; then

$$\sqrt{\frac{\sin(\beta - \alpha)}{\sin\beta\cos\alpha}} = 1.367$$

For simplicity we will assume that medium-speed runners ($\beta = 90°$) show a hydraulic efficiency of 84% (giving $\sqrt{e_h\, g} = 5.198$), and other types of runner an efficiency of 83% (giving $\sqrt{e_h\, g} =$ 5.167). These values are by no means taken too high for runners of fair design and construction. Then the speed constant has for the different types the following values:

Type of Runner	Speed Constant K_v
Low-speed ($\beta = 60°$ to $90°$)	4.588 to 5.198
Medium-speed ($\beta = 90°$)	5.198
High-speed ($\beta = 90$ to $135°$)	5.198 to 7.006

(4)

For very high heads, which naturally will require low-speed runners, it will be wise not to approach the minimum value of β, but to remain in the neighborhood of 90°, for the following reason: The smaller the angle β, or to be more exact, the smaller the ratio β/α, the smaller is the pressure-head under which the water passes from the guide case into the runner buckets. This reduced pressure will facilitate the separation of the air that is contained in the water, and thus it will facilitate honeycombing of the runner and guide case, so often observed even in turbines otherwise most carefully designed and highly finished.

Substituting the values of K_v in eq. (1) we obtain the following simple formulas for the runner diameters:

Type of Runner	Formula for Diameter
Low-speed	$\frac{87 \text{ to } 99}{N} \times \sqrt{H}$
Medium-speed	$\frac{99}{N} \times \sqrt{H}$
High-speed	$\frac{99 \text{ to } 134}{N} \times \sqrt{H}$

(5)

As far as speed alone is concerned, any diameter within the above wide limits could be used. The required capacity, however, will limit the choice considerably. The following considerations deal with the influence of capacity.

Let n = number of buckets.
n' = number of guide vanes.
t = thickness of bucket edge.
t' = width of the eddy caused by the guide vane tips and measured on the runner circumference (see Fig. 3).

Then the actual entrance area is:

$$B\left(\pi D - \frac{n\,t}{\sin\beta} - n'\,t'\right) = \pi\, B\, D\left(1 - \frac{n\,t}{\pi D \sin\beta} - \frac{n'\,t'}{\pi D}\right) = \pi\, K_1\, K_2\, D^2 \qquad (6)$$

where

$$K_1 = \left(1 - \frac{n\,t}{\pi D \sin\beta} - \frac{n'\,t'}{\pi D}\right) \qquad (7)$$

and

$$K_2 = \frac{B}{D} \qquad (8)$$

As to the number of buckets used differences of practice will be found among turbine build-

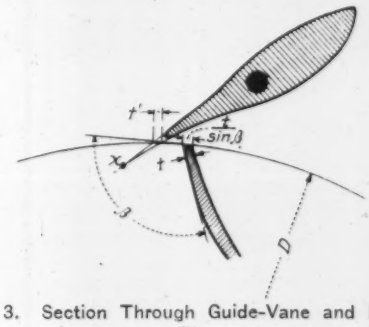


Fig. 3. Section Through Guide-Vane and Bucket Tip.

ers. While a few of them use in every case as large a number of buckets as possible, the majority put into a high-speed runner fewer buckets than into a low-speed runner. The following empirical formulas, in which D is expressed in inches, will give satisfactory results.

Type of Runner.	Approx. Number of Buckets.
Low-speed	$n = 3.7\sqrt{D}$
Medium-speed	$n = 3.0\sqrt{D}$
High-speed	$n = 2.2\sqrt{D}$

(9)

The number of guide-vanes is very often determined by the simple rule that in every case a few more guide-vanes than buckets should be

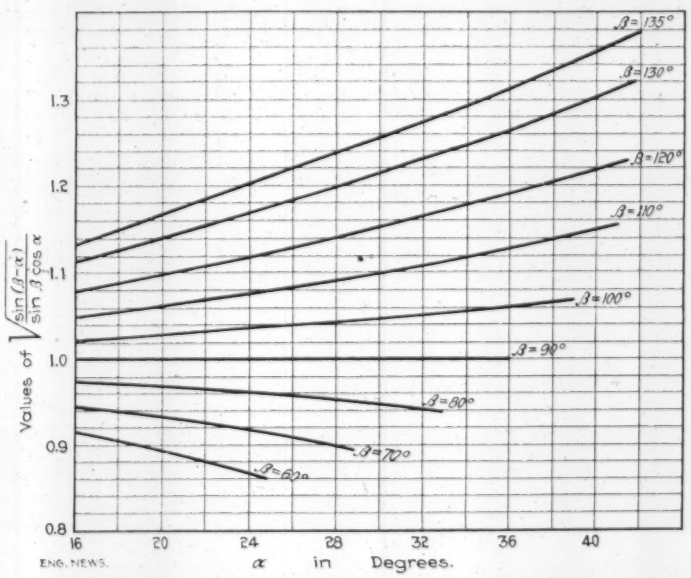


FIG. 2. CURVES FOR FINDING THE NORMAL PERIPHERAL SPEED OF A TURBINE RUNNER FOR GIVEN BUCKET AND GUIDE-VANE ANGLES.

The ordinates give values of

$$\sqrt{\frac{\sin(\beta - \alpha)}{\sin\beta\cos\alpha)}}$$

The speed-constant is

$$K_v = \sqrt{e_h\, g} \times \sqrt{\frac{\sin(\beta - \alpha)}{\sin\beta\cos\alpha}}$$

Where e_h = hydraulic efficiency.
g = gravity constant.

The desired peripheral speed of the wheel in feet per second is

$$v = K_v \sqrt{H}$$

where H = effective hydraulic head in feet.

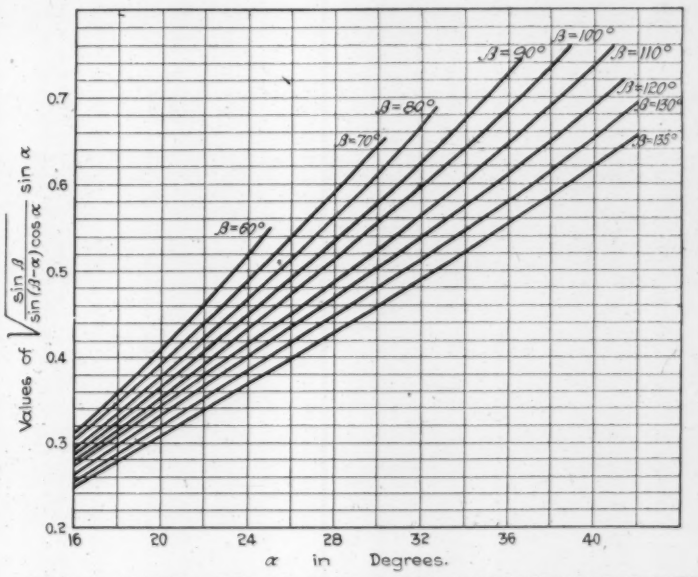


FIG. 6. CURVES FOR FINDING RADIAL ENTRANCE VELOCITY FOR GIVEN BUCKET AND GUIDE-VANE ANGLES.

The ordinates give values of

$$\sqrt{\frac{\sin\beta}{\sin(\beta - \alpha)\cos\alpha}} \cdot \sin\alpha$$

The factor $K_3 = \sqrt{e_h\, g} \cdot \sqrt{\frac{\sin\beta}{\sin(\beta-\alpha)\cos\alpha}} \cdot \sin\alpha$

where e_h = hydraulic efficiency.
g = gravity constant

The radial entrance velocity in feet per second is $c_r = K_3 \cdot \sqrt{H}$

where H = effective hydraulic head in feet.

put in (i. e., $n' = 1.1\ n$ to $1.3\ n$). This rule, however, gives low-speed runners too many guide-vanes, whereby (on account of the small angles α which are used in low-speed turbines) the gate openings become too small and correspondingly the frictional surfaces become relatively large. Therefore it is proper to take account of the guide-vane angle in choosing the number of vanes. The following empirical formulas will give good results. Again taking D in inches,

Guide-vane angle α.	Approx. Number of Guide-vanes.
20° and less	$n' = 2.5\ \sqrt{D}$
20° to 30°	$n' = 3.0\ \sqrt{D}$
30° to 40°	$n' = 3.5\ \sqrt{D}$

(10)

Since for manufacturing reasons it is advisable that the number of guide-vanes be even, and possibly divisible by four, it will be best to use an uneven number of buckets, in order to avoid having more than one bucket edge coincide with a guide-vane tip at the same time. On this basis the curves in Fig. 4 have been drawn. These may be used instead of eq. (9) and (10).

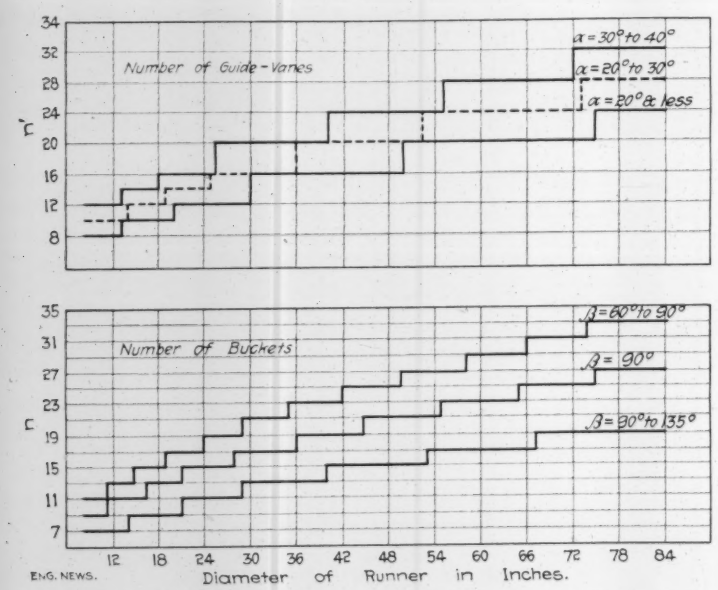

Number of Guide-Vanes
α = 30° to 40°
α = 20° to 30°
α = 20° & less
Number of Buckets
β = 60° to 90°
β = 90°
β = 30° to 135°
Diameter of Runner in Inches.
ENG. NEWS.

FIG. 4. DIAGRAM GIVING NUMBER OF BUCKETS AND NUMBER OF GUIDE-VANES FOR DIFFERENT TYPES AND SIZES OF RUNNERS.

The eddies caused by the guide-vane tips should be reduced to a minimum. The writer advises strongly to design the guide case in such a way as to get point x (see Fig. 3) outside of the runner circumference. This obviously must be obtained by shaping the vane tips properly and by leaving a sufficient clearance between vane and bucket tips. If this is done, the entering streams of water will join in a solid ring, and the effect of the eddies on the capacity of the runner will be nullified. The constant K_1 will then have the value,

$$K_1 = \left(1 - \frac{n\,t}{\pi\,D \sin \beta}\right) \qquad (11)$$

The thickness t varies between ⅛-in. and ¼-in. for steel plate buckets, and between ¼-in. and ⅜-in. for cast buckets.

The following table gives the values of K_1 for three different runner sizes, computed from eq. (11):

Runner Diam.	Bucket angle	No. of buckets	Constant K_1 — Steel plate buckets		Constant K_1 — Cast buckets	
D	β	n				
	60°	13	——			0.9154
1 ft.	90°	11	$t = \frac{1}{8}$ in.	0.9635	$t = \frac{1}{4}$ in.	0.9270
	135°	7		0.9671		0.9542
	60°	25	——			0.9600
4 ft.	90°	21	$t = \frac{1}{4}$ in.	0.9652	$t = \frac{1}{4}$ in.	0.9652
	135°	15		0.9649		0.9298
	60°	33	——			0.9557
7 ft.	90°	27	$t = \frac{1}{4}$ in.	0.9744	$t = \frac{3}{8}$ in.	0.9616
	135°	19		0.9745		0.9618

For simplicity we shall assume that K_1 has the uniform value 0.93 for all runner types and sizes, with the distinct understanding however that in the final computation the exact value is to be introduced, and also, if necessary, the item $n'\ t'$ be considered.

The capacity of the turbine depends very directly on the ratio B/D, or K_2. As a matter of fact it is this ratio which finally determines the limits for the application of radial inward-flow turbines. Turbine manufacturers are still struggling with the problem of extending these limits in both directions; therefore no definite maximum or minimum values can be given.

Present-day good practice indicates that until further advance is made it is safe to fix the limits of breadth of runner at 1/30 and ½ the diameter. The minimum value depends on the purity of the water. The maximum value which could be allowed depends on the design of the runner, for it is evident that the larger the width of the runner, the more difficult it is to secure the necessary passage area at the point where the water turns from radial to axial direction. In the opinion of the writer it is possible to go somewhat above ½ with the ratio of width to diameter, but then the runner must be bulged out sufficiently.

We have classified all runners under three types with reference to speed. It is customary to make a further classification with reference to capacity; here also we distinguish three types. The latter classification is based on the proportions of the runner profile, or the ratios of (1) diameter at entrance point of buckets, (2) diameter at exit point of bucket, (3) diameter of neck of draft-tube. These diameters (Fig. 5) will be denoted by D, D', and D''. Their ratios depend mainly on the factor K_2, or B/D.

Type I., or the low-capacity type, comprises all runners in which the draft-tube diameter D'' is less than the bucket exit diameter D', or at most equal to it, and in which B/D lies between 1/30 and ⅛. Type II., the medium-capacity type, comprises runners in which D'' is larger than D' but smaller than the entrance diameter D, and in which B/D lies between ⅛ and ¼. Type III., or the high-capacity type, comprises all runners in which the draft-tube diameter D'' exceeds the bucket entrance diameter D, and in which B/D is between ¼ and ½.

It is self-evident that high heads will require runners of both low-capacity and low-speed type, while low heads will call for high-capacity high-speed runners. In other words, small values of K_2 naturally go with small values of K_v, and large values of K_2 go with large values of K_v. Mistakes in this respect are frequently made by manufacturers of low-head turbines, when they occasionally build a high-head turbine, by giving the runner buckets of such turbines the same entrance angles ($\beta > 90°$) as are used on their low-head turbines. This increases the peripheral speed so much that the runner diameter, and consequently the size of the whole turbine, must be increased considerably in order to obtain the required rotative speed.

A runner is characterized as to its capacity by the so-called capacity constant.

$$K_q = \frac{Q_1}{D^2} = \frac{Q}{\sqrt{H}\,D^2} \qquad (12)$$

The values of Kq for the different runner types can be found as follows: Area × Speed = Discharge; therefore,

$$Q = \pi\ K_1\ K_2\ D^2\ c_r \qquad (13)$$

where c_r is the radial component of the entrance velocity c (see Fig. 1). This component is given by

$$c_r = c \sin \alpha = \sqrt{e_h\, g_H} \times \sqrt{\frac{\sin \beta}{\sin (\beta - \alpha) \cos \alpha}} \sin \alpha$$
$$= K_3\ \sqrt{H} \qquad (14)$$

Combining the last two equations, we get

$$Q = \pi\ K_1\ K_2\ K_3\ \sqrt{H}\ D^2 = K_q\ \sqrt{H}\ D^2 \qquad (15)$$

or,

$$K_q = \pi\ K_1\ K_2\ K_3 \qquad (16)$$

in which

$$K_3 = \sqrt{e_h\, g} \times \sqrt{\frac{\sin \beta}{\sin (\beta - \alpha) \cos \alpha}} \sin \alpha \qquad (17)$$

In Fig. 6 is drawn a series of curves which give the values of the second radical of eq. (17) for the same angles β and α for which the curves in Fig. 2 were drawn. Multiplying the appropriate ordinate taken from Fig. 6 by $\sqrt{e_h g}$, we obtain K_3. The other coefficients (K_1 and K_2) having been found previously, eq. (16) at once gives the value of the Capacity Constant K_q.

Using the same limiting values as before, to define the several types of runner, we find that the Capacity Constant has the following range:

Type of runner	Range of K_q
Low-speed low-capacity	0.21 to 0.89
Medium-speed medium-capacity	0.89 to 2.19
High-speed high-capacity	2.19 to 4.66

(18)

By introducing these values in eq. (12) and solving for diameter we obtain the following simple formulas:

Type of runner	Diam. in terms of discharge per 1-ft. head
Low-speed low-capacity	$(2.20 \text{ to } 1.06)\ \sqrt{Q_1}$
Medium-speed med.-capacity	$(1.06 \text{ to } 0.67)\ \sqrt{Q_1}$
High-speed high-capacity	$(0.67 \text{ to } 0.46)\ \sqrt{Q_1}$

(19)

These formulas, together with eq. (5), will determine which range of diameters can satisfy both the requirements as to speed and capacity. Evidently only those diameters are suitable which satisfy both eq. (5) and eq. (19).

The procedure can be further simplified by the use of a constant which the writer has called Type Characteristic (see Eng. News, Jan. 28, 1909). Its formula is:

$$K_t = \frac{N \times \sqrt{H\text{-}P}}{H\ \sqrt[4]{H}} = \frac{60\ K_v \times \sqrt{K_q} \times \sqrt{K}}{\pi} \qquad (20)$$

where

$$K = \frac{H\text{-}P}{Q\ H} = \frac{62.42 \times \text{turbine efficiency}}{550}$$

For a turbine efficiency of 80%, $K = \frac{1}{11}$, which may be used as a fair average value for the present purpose. With this figure, and the values of K_q and K_v, tabulated previously, we obtain the following ranges for the Type Characteristic:

Type of Runner	Type characteristic K_t
Low-speed low-capacity	12 to 28
Medium-speed medium-capacity	28 to 44
High-speed high-capacity	44 to 87

(21)

With these formulas the determination of proper runner type and diameter is very simple. We proceed as follows:

From the given values of horsepower output $H\text{-}P$, speed of revolution N, and hydraulic head H, compute the type characteristic K_t by eq. (20). If the resulting figure is between 12 and 28, a radial inward-flow turbine is possible, and the runner will have to be of the low-speed, low-capacity type, with $B/D = 1/30$ to ⅛, $\beta = 60°$ to 90°, and a profile which will fall between profiles A and B of Fig. 5.

If the value of K_t is between 28 and 44, the runner will have to be of the medium-speed, medium-capacity type, with $B/D = ⅛$ to ¼, $\beta = 90°$, and a profile which will fall between profiles B and C of Fig. 5.

If the value of K_t is between 44 and 87, the runner will have to be of the high-speed, high-capacity type, with $B/D = \frac{1}{4}$ to $\frac{1}{2}$, $\beta = 90°$ to 135°, and a profile which will fall between profiles *C* and *D* of Fig. 5.

If the value of K_t is smaller than 12, and it does not seem advisable to make B smaller than 1/30 of the diameter, a radial inward-flow turbine is not possible, and an impulse wheel will have to be used.

If the value of K_t is larger than 87, a multiplex turbine must be built. That is to say, a case for which K_t is found to be, say, 174, which is 2 × 87, would require a quadruplex turbine of which each runner is designed for $K_t = 87$.

Knowing the type of runner it is easy to find the other principal dimensions, as now we can not make a mistake in the choice of the rational values for the different constants in our equations. A few words must be added, however, in reference to the draft-tube diameter D''. The flow velocity c'' at the point where D'' is measured, the "upper draft-tube area," is, in properly designed runners, more or less the same as the flow velocity in the discharge area of the runner. This velocity represents a direct loss; but the loss is partly recovered by the conical lower part of the draft-tube. In low-capacity runners there is no difficulty in reducing the discharge loss to a minimum in the runner itself. In high-capacity runners, on the other hand, larger discharge losses must be allowed, as otherwise the runner would have to be bulged out too much. Expressing the discharge loss measured at the upper draft-tube area in parts of the total head, the following values will represent good practice.

Type of runner	Discharge loss in terms of total head	
Low-speed low-capacity	(0.04 to 0.06) H	
Med.-speed med.-capacity	(0.05 to 0.08(0.1)) H	(22)
High-speed high-capacity	(0.08 to 0.15(0.2)) H	

NUMERICAL EXAMPLES.—The following specimen calculations will illustrate the application of the method set forth in this article:

I.—Given $H = 100$ ft.; H-P = 2500 HP.; $N = 250$ r. p. m.

Assuming 80% efficiency, $Q = \frac{11 \times 2500}{100} =$ 275 cu. ft. per sec.

From eq. (20),

$$K_t = \frac{250 \times \sqrt{2500}}{100\ \sqrt[4]{100}} = 39.55$$

Comparing this with the sets of values given by eq. (21) we see that the runner has to be of the medium-speed, medium-capacity type, with bucket angle $\beta = 90°$. Therefore, from eq. (5),

$$D = \frac{99}{N} \times \sqrt{H} = \frac{99}{250} \times \sqrt{100} = 3.96 \text{ ft.}$$

Take $D = 4$ ft. The number of buckets is 21, from the chart Fig. 4. The thickness of bucket edge, using cast buckets, is ¼-in. The number of guide-vanes (from Fig. 4) is 20, but the guide case shall be designed in such a way that $t' = 0$ giving for the free circumference $\pi D - n t = \pi \times 48 - 21 \times \frac{1}{4} = 145.55$ ins. = 12.16 ft.

The type characteristic, 39.55, is nearer the upper limit for type 11 than the lower limit; assume, therefore, $B = \frac{1}{4} D = 1$ ft., and $D'' = D = 4$ ft. Then the free entrance area $1 \times 12.16 = 12.16$ sq. ft. Consequently,

$$c_r = \frac{Q}{12.16} = \frac{275}{12.16} = 22.62 \text{ ft. per sec.}$$

For $\beta = 90°$, $c_r = c = \sqrt{e_h g H}$. $\tan \alpha$. Hence, assuming $e_h = 0.84$, $\tan \alpha = \frac{c_r}{\sqrt{e_h g H}} = \frac{22.62}{51.98} = 0.43_5$ whence $\alpha = 23° 30'$.

The upper draft-tube area, if the shaft does not extend into the draft-tube, is $\frac{1}{4} \pi \times 4^2 = 12.57$ sq. ft. Consequently the discharge velocity is

$$c'' = \frac{275}{12.57} = 21.86 \text{ ft. per sec.}$$

and the discharge loss percentage is

$$\frac{(c'')^2}{2 g H} = \frac{22.86^2}{64.32 \times 100} = 0.0743.$$

which value is satisfactory, according to eq. (22).

II.—Given $H = 36$ ft.; H-$P = 4{,}000$ HP.; $N = 200$ r. p. m., so that $Q = 1{,}222$ cu. ft. per sec. It is required that the turbine be capable of sustaining 15% overload.

The type characteristic is

$$K_t = \frac{200 \times \sqrt{4000}}{36\ \sqrt[4]{36}} = 143.5$$

Comparing with eq. (21) we find that we must use a quadruplex turbine, with runners designed for

$$K_t = \frac{143.5}{2} = 71.8$$

This requires Type III, and consequently, by eq. (5),

$$D = \frac{99 \text{ to } 134}{N} \times \sqrt{H}.$$

Since the value of K_t is nearer the maximum than the minimum for Type III, take

$$D = \frac{125 \times \sqrt{H}}{N} = \frac{125 \times \sqrt{36}}{200} = 3.75 \text{ ft.} = 45 \text{ ins.}$$

Then the speed constant is

$$K_v = \frac{\pi D N}{60 \sqrt{H}} = \frac{\pi \times 3.75 \times 200}{60 \sqrt{36}} = 6.545$$

Assuming $e_h = 0.83$, $\sqrt{e_h g}$ is found to be 5.167. then,

$$\frac{K_v}{\sqrt{e_h g}} = 1.266 = \sqrt{\frac{\sin (\beta - \alpha)}{\sin \beta \cos \alpha}}$$

From the curves in Fig. 2, we see that we could use $\beta = 135°$ $\alpha = 31°$; or $\beta = 130°$, $\alpha = 35°$ 30′. We choose the former combination, because the turbine must be capable of carrying overload.

For these angles the value of $\sqrt{\frac{\sin \beta}{\sin (\beta - \alpha) \cos \alpha}}$ × sin α is 0.475, from Fig. 6. Consequently the radial entrance velocity $c_r = 0.475 \times \sqrt{e_h g H} = 0.475 \times 5.167 \times \sqrt{36} = 14.706$. The number of buckets, from Fig. 4, is 15. The guide case being designed so that $t' = 0$, the free circumference is

$$\pi D - \frac{n t}{\sin \beta} = \pi 45 - \frac{15 \times \frac{1}{4}}{\sin 135°} = 136.03 \text{ ins.} = 11.333 \text{ ft.}$$

Hence, $11.335 \times B \times 14.706 = \frac{1222}{4}$, whence $B = 1.823$ ft. Taking $B = 22$ ins., the ratio of width to diameter is $\frac{22}{45} = \frac{1}{2.045}$, which value is satisfactory.

Take into consideration the runner nearest the generator and assume that the shaft projecting from this runner into the draft-tube is 10½ ins. in diameter; then the upper draft-tube area in square feet is

$$\left(\frac{\pi D''^2}{4} - \frac{\pi \times 10\frac{1}{2}^2}{4}\right) \frac{1}{144}$$

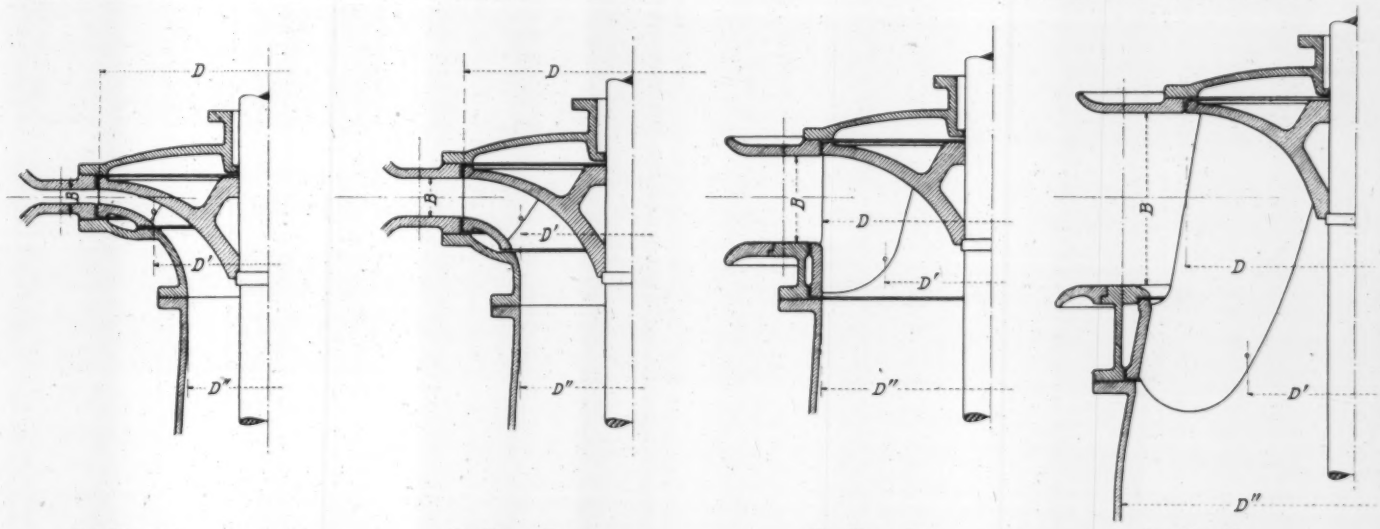
for D'' taken in inches. Assuming an allowable draft-tube loss of 0.14, we have

$$\frac{(c'')^2}{2g} = 0.14\ H, \text{ or } c'' = \sqrt{2g \times 0.14 \times 36}$$

= 18 ft. per sec.

the draft-tube velocity. Then the diameter of the draft-tube must be found from

$$Q = \left(\frac{\pi D''^2}{4} - \frac{\pi \times 10.5^2}{4}\right) \frac{1}{144} \times 18 = \frac{1222}{4}$$



Profile A — $\frac{B}{D} = \frac{1}{30}$; $D'' < D' < D$.

Profile B — $\frac{B}{D} = \frac{1}{8}$; $D'' = D' < D$.

Profile C — $\frac{B}{D} = \frac{1}{4}$; $D'' = D > D'$.

Profile D — $\frac{B}{D} = \frac{1}{2}$; $D'' > D$.

FIG. 5. LIMITING PROFILES OF THE THREE TYPES OF RADIAL INWARD-FLOW TURBINE.

Low-Speed Low-Capacity Type. *For High Heads.* $B/D = \frac{1}{30}$ to $\frac{1}{4}$. Bucket angle $\beta = 60°$ to 90°. Vane angle $\alpha = 20°$ and less

Medium-Speed Medium-Capacity Type. *For Medium Heads.* $B/D = \frac{1}{8}$ to $\frac{1}{4}$. Bucket angle $\beta = 90°$. Vane angle $\alpha = 25°$ to 32°

High-Speed High-Capacity Type. *For Low Heads.* $B/D = \frac{1}{4}$ to $\frac{1}{2}$. Bucket angle $\beta = 90°$ to 135°. Vane angle $\alpha = 30°$ to 40°

or

$$\frac{\pi D''^2}{4} = \frac{1222}{4} \times \frac{144}{18} + \frac{\pi \times 10.5^2}{4} = 2530.59 \text{ sq. ins.}$$

whence $D'' = 56.8$ ins. Using 57 ins. for round numbers, the discharge loss is reduced to $0.138\,H$.

The guide case and the regulating device shall be designed so that maximum gate-opening corresponds to the guide-vane angle $\alpha = 40°$. Assuming for the present that eq. (14) for radial entrance velocity is true up to the maximum gate opening we find with the aid of the curves in Fig. 6 that max. $c_r = 0.618\ \sqrt{c_0 g H}$, whereas at normal gate opening we found $c_r = 0.475\ \sqrt{c_0 g H}$. If we further assume that the efficiency remains unchanged, the discharge at maximum gate-opening 40° will evidently be

$$Q_{max} = \frac{0.618}{0.475} \times Q = 1.3\ Q,$$

or, in other words, the turbine is capable of 30% overload theoretically. Practically the overload capacity will be somewhat smaller, as the assumptions made are not correct. Formula (14) is valid only for the "normal" gate opening; i. e., that for which the turbine was originally designed and at which the water discharges from the runner buckets in planes going through the turbine axis. Further, the hydraulic efficiency always is smaller when the gate opening is different from the "normal" gate opening.

This is not the place to go any further into the difficult question of the variation of speed, discharge and efficiency with varying gate opening. Suffice it to state that, judging from actual tests, we may expect in the case at hand that the required overload, being only half of the overload as figured before, will certainly be obtained, very likely, before the gates are opened to the full 40° angle.

It is hoped that the above outlined method of computing water turbine runners will be useful to many engineers and will help to eliminate turbines of irrational types and proportions. It goes without saying that the values of the different constants indicated by the writer, must not be adhered to too closely, and that the boundaries between the different types are not sharp, but may be changed more or less according to the designer's preference. Thus it would not make much difference whether a runner of $K_1 = 43$ were designed with $D'' = D$, or with D'' somewhat larger than D.

AN EXPERIMENTAL BLAST with large, deep holes was recently made at the Warwick, Ohio, silica sand quarries of the Franklin Industrial Co. The quarry in question has a 40-ft. ledge of solid stone which heretofore has been shot down to crusher size by the short hole and light charge method, and the recent blasts were made to test the efficiency of the heavy shot method. For the first attempt, a series of four holes, 4 ins. in diameter and 40 ft. deep were drilled 13 ft. apart and 13 ft. back from the ledge face with a Cyclone drill. These holes were sprung with "dynalite" and then loaded with 275 lbs. of "dynalite" per hole. From the shooting of these four holes 1,387 cu. yds. of broken rock was thrown down, with an average size per piece of 50 lbs., exclusive of the very small material that could be easily shoveled. For this preliminary test the cost of drilling, by contract, was 30 cts. per ft., the cost of explosive, $171.87, or 16.8 cts. per cu. yd. of broken material for drilling and blasting.

On the second test three additional holes of the same dimensions were drilled. For each of these holes about 15 lbs. of dynalite was used in springing and 226 lbs. in shooting, a total of 726 lbs., at a cost of $90.75. From this shot a volume of rock 13 ft. back from the wall, 48 ft. long and 40 ft. deep was thrown down, a total volume of 927 cu. yds. The cost of this second drilling was 30 cts. per ft., or 3.9 cts. per cu. yd. of material. This, with the 9.8 cts. per cu. yd. for powder makes a total of 13.7 cts. per cu. yd. for the second test.

The powder used in the tests cost 12½ cts. per lb. and was tamped with sand to a height 8 ft. below the top. It is thought that the method is advantageous both from a standpoint of economy and because the material is broken up in smaller pieces so that it can be passed to the crusher with less intermediate breaking. If the method is to be used continuously the cost of drilling can be lowered.

A Composite Sand and Rock Foundation for a Tall Building.

An unusual situation developed recently in the construction of the foundations of the Municipal Building of New York City, an office-building which is to have 25 stories capped with a tower rising 560 ft. above street-level. It has been decided that, while two-thirds of the building will be founded on bed rock, the remaining part must be founded on sand. The proposal to do this aroused active comment among the public, and something of a controversy developed over the matter. However, the engineers in charge felt no great uncertainty about the advisability of the step, and on Thursday last, Dec. 30, they secured the approval of the city's Board of Estimate to modifying the foundation contract accordingly. This approval was one of the last acts of the Board, a new Board coming into office on Jan. 1.

The following is a short account of the conditions which led to the action noted.

The Municipal Building was designed by the architectural firm of McKim, Mead & White, who were awarded first place in a competition of plans. It is to house the principal offices of the city departments (most of which now occupy rented quarters), and in its basement will accommodate a six-track station of the new Bridge Loop Subway. The site is only a hundred feet north of the Manhattan terminal of the Brooklyn Bridge and just east of the Hall of Records, County Court House and City Hall. It was previously occupied by ordinary store buildings of three to six stories. The plot of ground is trapezoidal, about 400 ft. long by 200 ft. wide, extending from the intersection of Center St. and Park Row north to Duane St. It is crossed from east to west by Chambers St., but the building will span this street, and in fact has its high tower directly over the arcade through which Chambers St. will pass.

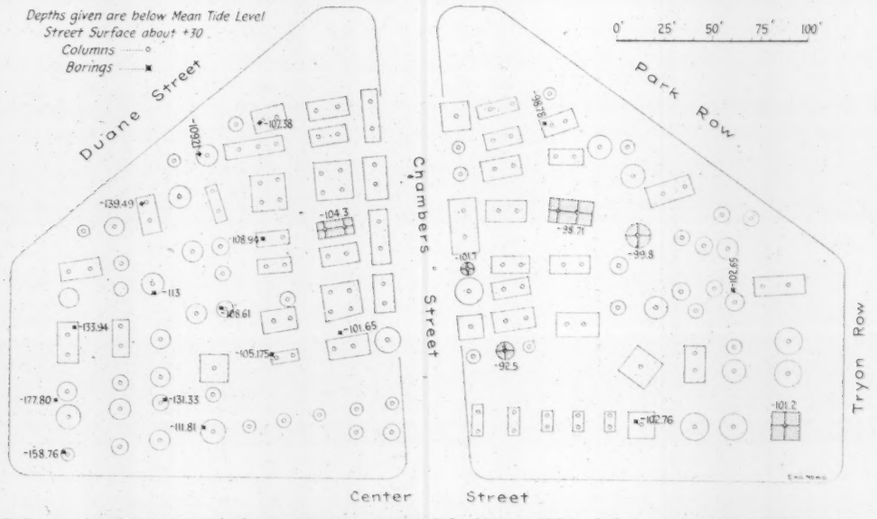


FIG. 1. CAISSON PLAN OF MUNICIPAL BUILDING, NEW YORK CITY, SHOWING DEPTHS OF CORE BORINGS AND COMPLETED CAISSONS.

(The six shaded caissons have already reached rock, with depths as noted. All depths are below mean tide level, which is about 32 ft. below street level.)

The project to erect this building has been on foot for nearly half a dozen years. When the scheme took definite form, an act of the State Legislature placed the work of construction in the hands of the city's Bridge Department. The effect of this step was to render the city Building Code of no application to this particular building, as will appear. The statutory limitations on stresses, loads, materials, etc., were thus removed, and the design was subject to the free discretion of the Department of Bridges and the architects, under the sole limitation that all plans be approved by the Board of Estimate.

PLAN FOR ALL-SAND FOUNDATION.—The original design of the foundations was based on information obtained from four wash-borings put down for the Department of Bridges. These borings indicated rock at a depth varying probably from about 94 ft. to 103 ft., sand overlying the rock throughout. The depth to rock was near the limiting depth for compressed-air work, and the task of founding on the rock by concrete piers sunk by the pneumatic-caisson method, the standard for tall buildings in lower New York, seemed so difficult and costly so as to make it advisable to found directly on the sand. At least the architects and the engineers of the Department of Bridges were of this opinion.

They had good precedent for sand foundation in at least two high and very heavy buildings located close at hand. The St. Paul Building, a 26-story steel-frame structure only 1,000 ft. to the southwest, rests on footings in firm sand, and has stood for a dozen years without any indication of defect in its foundations. The World Building, a heavy bearing-wall structure 20 years old, also on sand, is not more than 300 ft. from the site, and here too no weakness in the foundations has become apparent. Both these buildings are subject to the vibration of the Rapid Transit Subway, which passes directly in front of them. The Municipal Building also adjoins this subway, besides having the Loop Subway pass through its basement.

It was therefore decided that footings in the sand would be safe and satisfactory, provided they were carried down to a level well below all chance of future disturbance by other excavation work in the vicinity. With this object, plans were made for concrete piers going down 28 ft. below mean tide level, the latter being just below the basement or track floor level. These piers were to be put down by means of pneumatic caissons. Their dimensions were fixed on the basis of a permissible bottom pressure of 8 tons per sq. ft.

THE BUILDING CODE NOT APPLICABLE.—Bids were called for on this foundation. J. H. Gray & Co. were the lowest bidders. Before the contract could be executed, however, a court injunction was obtained on the plea that the plans were not in accordance with the law, the Building Code limiting the pressure on sand foundations to less than half the figure here used (on firm coarse sand the Code allows 4 tons per sq. ft.; in this location much fine sand is found.) When the case was heard, the court decided that neither the Building Code nor the Bureau of Buildings (which administers the building laws) have anything to do with the Municipal Building. This cleared the way for executing the contract.

ROCK FOUNDATIONS ADOPTED.—But now the Board of Estimate stepped in, under the stimulus of strong agitation against the proposed

foundation on sand, and directed that the plans be altered to provide for a foundation on bedrock.

Retaining the general features of the foundation, the plans were altered for caissons going down to rock, and the caissons were decreased in size to make the pressure on the rock 15 tons per sq. ft. New bids were called for and the lowest bidder, The Foundation Co., was awarded the contract and soon began work.

In planning the details of his work, the contractor proceeded to put down several core borings on the site, in order to determine definitely the character of the materials passed through and the depth at which rock would be encountered. The first two holes, both in the northerly half of the plot, showed depths of 131 ft. and 177 ft. to rock, and, remarkably enough, the deeper of the two borings was only a short distance from one of the wash borings originally made by the Department of Bridges, in which a depth of 94 ft. had been found. That wash boring apparently had brought up on a boulder and thus failed to give the true depth. The core borings confirmed the wash borings, however, in showing that the material overlying the rock was sand of varying fineness, at some points containing boulders.

ROCK BELOW THE PNEUMATIC LIMIT.—The great depth shown by the core borings at once made it seem impossible to reach rock by the pneumatic process. The sand was so fine in the lower parts as to make it probable that air pressures equal to the full hydrostatic head would have to be employed in sinking the caissons. As pneumatic work finds a practical limit at 50 lbs. per sq. in., corresponding to depths of 110 to 115 ft., it was not clear what could be done to carry out the contract on the original basis. The findings were therefore at once reported to the Department of Bridges, and the latter undertook a more extensive set of core borings, to explore the site in its entirety and furnish all information necessary for a decision. These borings are not yet concluded. Up to date 16 holes have been put down, with the results shown on the plan reproduced herewith as Fig. 1.

The new set of borings soon made it evident that, while the depths originally expected prevailed over the greater part of the site, the rock surface in the northerly portion sloped downward to a maximum depth of 175 to 180 ft. at the northerly line of caissons. (The wash boring which showed 94 ft. was about 40 ft. north of this line.) To carry on pneumatic work at the depths found in this northerly part was judged out of the question.

A canvass of other means of carrying the work through was now made. Attempts to pump down the groundwater level in connection with pneumatic caisson work were considered too hazardous to be tried. Consideration was given to consolidating the subsoil (1) by freezing and (2) by injecting cement grout. The former was considered far too costly and uncertain in proportion to the advantages to be derived. Cement injection, it was thought, would not prove successful in the fine sand strata which would have to be pierced. Other means to penetrate below the depths which could be reached by the pneumatic caissons, such as dredging in open cylinders, all seemed to involve the risk of subterranean flow of material, and this risk, in view of the existence of numerous important structures in the neighborhood at higher levels, was considered prohibitive. Such methods as sinking pipes from the bottom of pneumatic caissons, to obtain virtually a concrete pile foundation, were not regarded as capable of giving a thoroughly reliable foundation.

A survey of the situation on these lines led to the conclusion that the simplest, if not the only, way out of the difficulty was to retain the old scheme of a sand foundation for that part of the building where the rock was too deep to be reached by the caissons. The two disadvantages of possible settlement and possible future disturbance were regarded as negligible. In the first place, the sand at a reasonable depth below water level would be secure against settlement provided the foundation pressures were moderate. In the second place, if the footings were carried down to a depth well below the level of any probable future excavation in the neighborhood there could be no chance for disturbance, in the ordinary course of things.

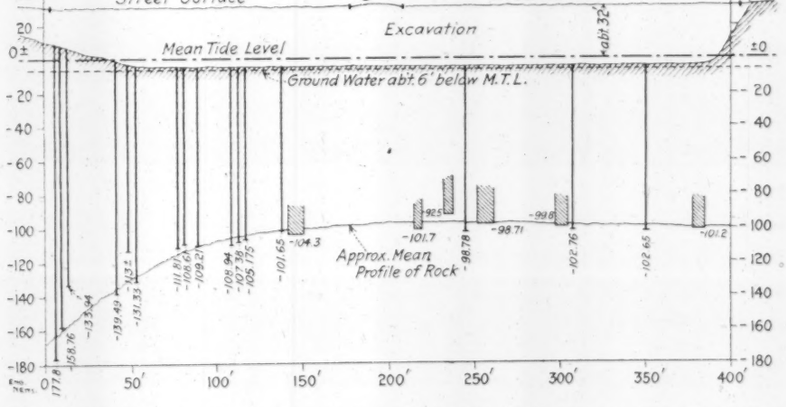


FIG. 2. APPROXIMATE ROCK PROFILE IN NORTH-SOUTH SECTION OF SITE OF MUNICIPAL BUILDING, NEW YORK CITY.

(Core borings are shown by heavy vertical lines. Caissons sunk to rock up to Jan. 1, 1910, are shown by cross-hatched rectangles. The four washborings made before designing the foundations are not indicated. Two of them, at Chambers St., and one near caisson 300 in the above profile, gave a depth practically the same as above shown for rock. The fourth would be located 50 ft. to the left of the diagram; it showed rock at —94 ft. and is thought to have struck a boulder. The core borings were required to give 10-ft. cores of rock to avoid the danger of showing a boulder as bed rock.)

RETURN TO SAND FOUNDATIONS FOR PART OF THE BUILDING.—Carrying out this idea, the caissons in the northerly part of the site were redesigned for a depth of 40 ft. below mean tide level and a bottom pressure of 6 tons per sq. ft. (as against the original scheme of 8 tons on sand and the later plan of 15 tons on rock). Fortunately the structural arrangement of the building permitted a rather natural division between the rock-founded and the sand-founded portions of the building. Roughly speaking, the structure is divided into thirds, the tower being the central third, flanked by northerly and southerly wings. Pneumatic foundations to rock were considered feasible throughout the tower and the southerly wing and part of the northerly wing. The northerly wing in general would have to rest on sand, however. The division was therefore made complete by a line drawn between tower and north wing, everything south of this line to be on rock and everything to the north on sand.

The foundation plan Fig. 1 exhibits the caisson layout as prepared for the rock-foundation contract, and shows in addition the depth of all core borings made prior to Dec. 31, 1909, and the depths of the six caissons which at that date had reached rock, a total of 22 recorded depths. The information as to depths given by Fig. 1 has been plotted in profile in Fig. 2, which represents an approximate north-and-south section through the site. The profile of rock surface as drawn in Fig. 2 is an attempt to average the depths shown by the various holes. It indicates clearly a pronounced dip in the rock surface, beginning not far north of Chambers St. and continuing to the northerly limit of the site.

Referring again to Fig. 1, the rectangular belt of large caissons enclosing Chambers St. indicates the outlines of the tower portion of the building. Drawing a transverse line just to the north of this separates off the north wing, which under the new plan is to be founded on sand (the group of small circular caissons along Center St. just north of Chambers St. will also have sand foundation).

CHANGING THE CONTRACT.—The necessity for an explicit modification of contract to authorize sand foundations on part of the site arose from the wording of the contract entered into with The Foundation Co. Quoting from this contract:

> The specifications are intended to cover all material and labor required to complete the caisson foundations to the levels indicated on the drawings. . . . The caisson foundations are to be of concrete as shown on the caisson plan, and to be put in place by the pneumatic process. All caissons must be carried down to rock. . . .

When this contract was entered into, both parties tacitly assumed that the wash borings truly indicated the depths which would be met with. The contract drawings correspondingly showed the caissons resting on rock at the approximate depths indicated by the borings. How to construe such a contract in a case where part of the work could not be gotten down to rock by the pneumatic process seems rather puzzling, and a change in the contract was unquestionably the simplest way out of the difficulty.

It will be readily understood that the contractor's estimate did not include a margin for experimenting with new methods to reach greater depths than heretofore considered practicable. He might therefore be morally as well as legally justified in declaring that, if the caissons themselves could not be sunk to rock, he was not required under the contract to first sink caissons to the greatest depth possible under this process, and then sink foundations of another kind from the caisson floor down to rock; or to devise means by which the range of the ordinary pneumatic-caisson method could be extended to unprecedented depths.

It is probable that with this difficulty in view, the authorities decided upon the composite sand-and-rock foundation as the readiest adjustment of the contract under the circumstances. However, engineers of the Department of Bridges have also stated that no method of construction is available by which satisfactory foundations could be sunk to rock, unless at extreme risk of failure and of disturbance of adjacent structures.

COST OF THE FOUNDATION.—The contract price for the original sand-foundation project (J. H. Gray Co.) with bottom level at — 28 ft. and bottom pressure 8 tons per sq. ft. was $622,000. This covered the work of sinking 125 caissons.

The rock-foundation contract (Foundation Co.) comprised 116 caissons (some of the former ones being combined). The contract provided a lump sum for foundations to — 96 ft. and prices per cubic yard for work below — 96 ft. The lump sum was $1,443,147, or roughly $820,000 more than the sand-foundation contract.

The present modified plan takes away from the rock-foundation scheme 48 caissons (all north of Chambers St. except twelve large caissons in a double row along the north edge of Chambers St., these being part of the tower footing; see plan Fig. 1), and substitutes for them 38 caissons of larger size (proportioned for

6 tons base pressure) at — 40 ft. base elevation. The cost of the eliminated caissons was estimated at $347,916, and the cost of the substituted ones at about $6,000 less. Both sides agreed to contract for the modification at the original price, the contractor offsetting his claims for delay, etc., against the slight reduction of estimated cost.

These figures do not include the price for various accessory work, excavation, etc., which is identical in all three contracts. Further, the ultimate cost of the foundation will be increased by the payment for work below — 96 ft., as all of the caissons to rock will go somewhat below this level so far as the present borings indicate.

Annual Convention of the American Society of Agricultural Engineers.

Reported by CHESTER O. REED.*

The third annual convention of the American Society of Agricultural Engineers was held at the Iowa State College, Ames, Ia., Dec. 28 and 29, 1909. That agricultural engineering has become a profession and that the society has undertaken a long-needed work and entered a hitherto neglected field, becomes more and more apparent as the society grows older and with precaution and wise judgment begins to gnaw at the problems that are confronting the manufacturer, farmer and teacher on the mechanical side of rural life. It may be stated that the term agricultural engineering embraces a study of farm machinery, farm buildings, irrigation, drainage, and rural road building—involving manufacturing, teaching, and experimental activities along these lines.

The sessions were attended by prominent manufacturers, men engaged in teaching agricultural engineering in our several universities, experimentalists, and practical farmers. The papers presented showed clearly the vastness of the work that is being undertaken and give an intimation as to the extent of the field for such work and the benefits to be derived by such a cooperation of these men as is fostered by the society.

Many discussions of interest to engineers were entered into, bringing to light intricate problems that remain to be solved. A paper prepared by Mr. A. O. Fox, of Madison, Wis., on the "Standardization of Farm Machinery," was the key that opened the gate to a new field of discussion. Mr. Fox strongly advocated that the society should, after most careful study, deliberate investigation and experiment, set standards for farm machines and machine parts that the manufacturers must meet before their product can be sold under recommendation of the society. This would result in the protection of the farmer and the manufacturer alike. But little reason could be given why farm machinery, in which such vast amounts of capital are expended yearly, should not meet standard requirements in construction, and why the farmer should not be protected, in buying, as well as the manufacturer who purchases a standard machine recommended by an engineering society. The manufacturers present could not endorse such action on the part of the society too highly. They claimed that it was what the manufacturers of farm implements were waiting for, as Mr. Blake's paper had suggested. The tremendous amount of work and responsibility involved if such action should be taken by the society was carefully discussed and methods of procedure were suggested. The matter was left with a committee which is to report on methods and make recommendations at the next annual meeting.

An interesting paper on "A New Six-Stroke Cycle Engine" was presented by M. L. King, the Experimentalist at the Iowa State College. In this engine the water used in cooling, by entrance to the cylinder, is turned to steam, expands, and does work in the fifth cycle. An extract from Mr. King's paper gives a general idea of the principle of the engine.

> The first three strokes are similar to the first three strokes of the Beau de Rochas or Otto four-stroke cycle. In the fourth or exhaust stroke, the exhaust valve closes in time to permit the admission of a small quantity of mixed water and steam coming directly from an exhaust heater. This fluid absorbs heat from the cylinder walls, piston head and piston rod in case of the double-acting engine, expands, doing work in the fifth stroke, and is exhausted, thus scavenging the cylinder, during the sixth stroke, which is the last event of the cycle.

Such an engine is to be constructed by Mr. King during the ensuing year at the Iowa State College. It is claimed to have advantages over the four-cycle machine in cooling, uniformity in mixture, governing the variable cut-off and in the fluctuations.

Mr. J. T. Stewart, Agricultural Engineer of the University of Minnesota (St. Paul), read a valuable paper on "Roads," in which he advocated strongly the cash system of maintenance to take the place of the old labor system. His arguments against the labor system were:

(1) There is no general plan of work.
(2) The roads are not worked at the proper season of the year.
(3) A large amount of money is invested in machinery which stands idle during a great part of the year.
(4) Much time is lost by inexperienced persons using road machinery.
(5) Repairs are not made when necessary.

The great loss to the farmer by having to contend with poor roads and the loss to the entire community, in fact, was forcibly brought out by startling statistics and instances of individual loss. In conclusion Mr. Stewart said:

> The history of roads shows that they pass through a state of development, and ordinarily this general development should be encouraged and not let come to a stand; the principles of road building should be clearly taught as adapted to local conditions; drainage should be promoted before roads, and thoroughly explained, and the cash system should replace the labor system; the farmer element should be taught that their roads can be bettered with the material and money at hand, and facts should be clearly brought out that good roads can be had in nearly all localities by sandwiching some local material between good drainage and eternal vigilance in maintenance.

Dean Marston, of the Iowa State College, spoke at some length on the disposal of farm sewage. Inasmuch as the Experiment Station of that college is soon to publish a bulletin on this subject, the Dean did not present a paper, but in his remarks he strongly advocated the double-tank system, with intermittent deliveries to the sand filter before the water is set free.

Discussions of new formula for finding the horse-power of traction engines were led by Mr. Wallace F. MacGregor, of the I. J. Case Manufacturing Co., Racine, Wis., and new formulas for finding the capacity of tile drains were presented by E. W. Hamilton, of the Iowa State College.

Besides the matter on formulas Mr. MacGregor's paper included a brief historical review of traction engines, presented the characteristics of the ideal traction engine and dealt at some length with the traction-engine design and construction of to-day. A companion paper dealt in much detail with the traction plow. This paper was by Mr. E. W. Hamilton, Editor of the "Canadian Thresherman and Farmer, Winnipeg, Man.

The advisability of the society taking charge of the annual motor contests that are held in the States and in Canada was discussed, and the general expression favored such action. Further action in the matter was left to a special committee empowered to act.

During the business session the following officers were elected:

President.....P. S. Rose, Madison, Wis.
1st Vice-President,
Prof. M. L. King, of the Iowa State College, Ames, Ia.
2d Vice-President,
J. B. Bartholomew, Pres. Avery Mfg. Co., Peoria, Ill.
Secretary,
Prof. E. W. Hamilton, of Iowa State College, Ames, Ia.
Treasurer,
E. W. Hamilton, Editor of Canadian "Thresherman and Farmer," Winnipeg, Man.
Chairman of Research Committee,
C. I. Gunness, of the N. Dak. Agricultural College, Fargo, N. Dak.

President Rose was formerly of the North Dakota Agricultural College, and is now with the Gas Review and American Thresherman publications. A banquet tendered by the management of these magazines ended the sessions.

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The Proposed Regulation of the Level of Lake Erie.

Many of our readers are more or less familiar with the plans which have been under consideration for some ten years past to regulate the level of Lake Erie by the construction of a dam and sluice-gates at the head of the Niagara River, through which Lake Erie discharges into Lake Ontario.

The investigation of this project is in the hands of the International Waterways Commission. The annual report of that body to the Secretary of War, just made public, opposes the plans previously made for regulating the level of Lake Erie and advises instead that the channel of the Niagara River be restricted so as to raise the level of Lake Erie without decreasing the fluctuations between high and low water level. The reasons for this recommendation are given in the following extract from the Commission's report:

> The study of the regulation of Lake Erie was continued during the year. The mutual interdependence of the Great Lakes and their connecting channels is so close that a study of one is not complete without a study of all. The effect of an attempt to regulate Lake Erie cannot be fully ascertained without an hydraulic analysis of the general regulation of all the lakes. This analysis was a long and laborious undertaking, but it was practically completed early in 1909. It includes a large number of tables and plates, many of them of an elaborate character.
>
> A study of these documents led to the conclusion that it was not practicable to regulate the monthly mean level of Lake Erie within a range of 1 ft., as proposed by one of the boards of engineers which preceded this commission, but that it would be practicable to regulate it within a range of about 20 ins. A further study of the effect of such regulation led to the conclusion that while it would benefit the navigation of Lake Erie, Lake St. Clair, and Lakes Michigan-Huron, it would injure Lake Ontario and the St. Lawrence River, and the low-lying shores of Lake Erie, especially at Buffalo.
>
> If the advantages and disadvantages could be equally distributed among all the persons affected the former might outweigh the latter, but as a matter of fact those who navigate the St. Lawrence canals are not specially concerned with deepening the harbors of Lake Erie, nor are those who occupy the low-lying portions of Buffalo and other places sufficiently compensated for the injury to their property by the beneficial effects upon navigation. Questions of a peculiarly intricate character would arise of damages to vested rights. It seemed to the commission that the advantages to be derived from this regulation were not of such overwhelming character as to justify the two governments in raising these vexatious questions.
>
> By the term "regulation" as here employed is meant the most complete practicable regulation, such as can be secured by a dam and sluice gates located at or near Buffalo. While it may be inexpedient to construct such works, it does not follow that nothing can be done to improve or maintain the level of the lakes. It is possible to raise the level of any lake by simply reducing the size of its outlet. With a reduced cross-section the outlet requires a steeper slope, and the average level of the lake is raised, but the oscillations will go on as before, and the discharge will remain the same. To raise the level of Lake Erie will raise also, but to a less degree, the levels of Lake St. Clair and of Michigan-Huron, and will thus benefit those waters, while it will have no effect on Lake Ontario or the St. Lawrence River.
>
> It is believed that somewhere in the Niagara River between Lake Erie and the Falls, a submerged dam may be placed which will greatly benefit the navigation of the waters above, without injury to those below, and with only minor damages, if any, to the adjoining land. The upper river is a valuable safety valve for the protection of Buffalo in time of storms upon Lake Erie and should not be obstructed by a dam. It is possible that the extreme lower end of the reach, that is, the section just above the Falls, may not be available because of excessive overflow to be caused in the valley of the Welland River. To determine the best site it has been necessary to make additional surveys. These were begun in July, 1909, and are still in progress.

VALUE OF SAND AND GRAVEL.—It is a surprising fact that sand and gravel taken from the Pasig and Mariquina rivers, near Manila, P. I., have sold at the dredges for more per cubic meter than the average value of the gold per meter in good gold dredging ground, and that the price of crushed stone at the quarries has in some cases been so high as to make the quarrying compare favorably, considering the expenses of production, with the mining of low grade ores.—The Philippine "Journal of Science," September, 1909.

A BOILER EXPLODED at West Reading, Pa., Dec. 29 at the new power plant of the Metropolitan Electric Co. Five men, in the employ of the contractors erecting the power plant, were killed. The boiler, vertical fire-tube, was used to furnish steam for driving a concrete mixer. The accident occurred at about seven in the morning just as the men were about to begin work.

BY THE WRECKING of an aeroplane, Leon Delagrange was killed at Bordeaux, France, on Jan. 4. He was using a small monoplane of the Bleriot design, which he had equipped with a gasoline motor of much greater power than it had before been considered advisable to use on this light flyer. The use of this engine is supposed to have caused the wreck, in some way. The aeronaut made his last ascent in a strong wind and when about 65 ft. above the ground one wing broke, it is reported, and the other collapsed. Delagrange was made prominent by his daring exploits with aeroplanes, having held, at various times, records for speed and distance, as has been noted in these columns. He is the fifth prominent aviator to be killed since purely mechanical flight became possible—Lieut. Thos. Selfridge, Sept., 1908; E. Lefebvre and Capt. Louis Ferber, Sept., 1909; A. Fernandez, Dec., 1909.

A BUTTING COLLISION AT OXFORD, MONT., on the Great Northern Ry., Jan. 1, killed three trainmen when passenger train No. 44 met an extra freight. The accident took place during a blinding snow-storm.

AN EXPLOSION IN A MINE near Centralia, Ill., Dec. 27, caused the death of four men employed as shot firers. The explosion, evidently of dust or gas, occurred when one of the shots was fired. The mine, which is owned by the Centralia Coal Co., was heavily damaged by the explosion.

The explosion of 150 lbs. of dynamite at the Ollie Bell mine, near Mineral Point, Wis., Dec. 28, killed four miners. The men had entered the powder room in a drift near the top of the main shaft for the purpose of returning some unused blasting powder.

AN EXPLOSION OF PINTSCH GAS at Montreal, Que., Dec. 31, wrecked the Canadian Pacific train platform at Place Vigor station in the eastern part of the city. Twenty-two persons were injured. The platform was built of heavy planks and beneath it ran a pipe used to supply gas to the tanks of passenger coaches. The accident is ascribed to the accumulation of gas leaking from this pipe beneath the platform, whose cracks were stopped up by snow and ice.

AN EXPLOSION OF GRAIN DUST in the plant of the Buffalo Cereal Co., Buffalo, N. Y., seriously injured nine men and is supposed to have killed three others who are missing. The explosion occurred in the yellow corn mill which was wrecked, together with a concrete and brick elevator adjoining it. The wreckage of both buildings caught fire and the flames spread to other parts of the plant.

SPREADING RAILS CAUSED A WRECK on the Chicago, Rock Island & Pacific Ry., Dec. 31, three miles south of Trenton, Mo., according to the decision of a coroner's jury. Three persons were killed, two of whom were passengers, and nine others were severely injured. The train was a westbound passenger known as the California Special. The locomotive and all but two of the cars were derailed and broken up. A fire started in the wreckage and several passengers caught in the wrecked cars had narrow escapes from death by burning.

A REAR COLLISION AT CROTON, N. Y., Dec. 31, on the New York Central & Hudson River R. R. resulted in the death of Mr. Spencer Trask, a prominent capitalist of New York City, and the injury of several other persons. The second section of the Montreal express had been stopped in Croton by a block signal. A fast freight following the express at a speed of about 30 mi. per hr. crashed into it, practically demolishing the rear sleeper and the rear portion of the second sleeper. Mr. Trask was in the rear sleeper. The accident occurred shortly before eight o'clock in the morning and on a long stretch of straight track.

The engineer of the freight, according to newspaper reports, admitted that the block signal immediately behind the express was set at "danger," but declared that the preceding signal stood at "safety" instead of at "caution." The annual report of the Block Signal and Train Control Board to the Interstate Commerce Commission, dated Nov. 22, 1909, states that the New York Central is replacing with automatic signals the controlled manual block-signal system on its main line between New York City and Buffalo. We quote from the above-named report as follows:

Several sections, aggregating about 50 miles of line, have been changed already, and it appears to be the intention to supersede the manual by the automatic on the whole of the 440 miles between the cities named, the larger part of which is four track line. The chief reason for making the change is to increase the capacity of the road by making the block sections very much shorter than heretofore. To do this with manual signals would increase the annual cost by the amount of the wages of the three signalmen [succeeding each other in eight-hour shifts—Ed.] required at every new block station; whereas with the automatic signals, requiring no attendants to operate them, the cost of operation is only slightly increased by increasing the number of blocks.

Reports state that the new signals in use at Croton at the time of the wreck had been in operation only a few weeks.

DYNAMITING A RAILROAD was attempted on a large scale by a gang of men in Baltimore, Md., it is charged by police officials of that place. An explosion occurred under the Baltimore & Ohio Ry. bridge at Gay St., Baltimore, on Dec. 31, without doing great harm. Later several men were arrested while approaching the Mount Clare shops of the railroad company with dynamite in their possession. It is charged that a plot had been formed also to blow up a bridge on the railroad outside of Baltimore. Two months ago an attempt to dynamite a roundhouse at Baltimore was discovered. These and other acts of violence are charged against various members of a gang of six or eight men who have been arrested within the last week. Convictions are likely. This case is noteworthy as being virtually the first instance among the score or more dynamite outrages which occurred during the past two years in connection with contract work, where any success has been achieved in apprehending the criminals. It is also noteworthy because hitherto the dynamiting has been restricted to contract work, chiefly steel bridge and structural work, and completed structures already in public use were not attacked. The present case is the first important one in the series, in which dynamiting has been directed against structures in regular use.

THE MANHATTAN BRIDGE over the East River at New York City was formally opened on Dec. 31. by the Mayor and the Bridge Commissioner of New York City. The superstructure of this bridge has been designed and erected wholly in one city administration, the one which just went out of office. Although some of the work on the decking was not yet completed, the vehicle roadway is in shape for use, and the officials desired to record their connection with the undertaking by an inaugural during their term of office. The bridge as a whole has been under construction for a longer period, however, the first substructure contracts being let in 1901. The bridge is a wire-cable suspension structure of 1,470-ft. span c. to c. of main towers. It is the fourth of the large East River bridges to be opened. A full investigation of the design and construction was conducted within the past year by Mr. Ralph Modjeski, with the result that the structure was fully approved in all details (see Eng. News, Oct. 14, 1909, pp. 401-409).

AN ABRASIVE MATERIAL known as "Corubin" is obtained as a by-product in the production of chromium by the "Thermit" process. In producing chromium by this process, chromium oxide is mixed with finely divided aluminum and the whole is ignited. In the ensuing chemical reaction, the aluminum combines with the oxygen of the chromium oxide and sets chromium free. A slag is formed consisting of aluminum oxide with traces of chromium in it. It is said to be entirely free from contained moisture. This slag, called "Corubin," has been found to be an excellent abrasive and has been for some time in use in Europe in the form of grinding wheels, emery cloth, etc. The Goldschmidt Thermit Co., 90 West St., New York City, is now in a position to supply this abrasive to the American market.

A WOODEN WATER-TANK at Glencoe, Minn., failed on Nov. 25, 1909, through weakening of the bands by corrosion. The tank formed a part of the water-works system built about 1897. The tank was 30 ft. in diameter and 22 ft. high, built of 3-in. staves held in place by flat bands spaced about 6 or 8 ins. Mr. C. V. Corson, City Clerk, informs us that the wood was sound in every particular, but that the "bands rusted from the inside." Fairbanks, Morse & Co., of St. Paul, who were contractors for the Glencoe water-works, state that the failure was due to lack of inspection or care of the tank, which they say had received no attention since it was built 12 years ago. Mr. Corson states that the tank will be replaced by a new one, also of wood, but with round instead of flat bands and with an inspection balcony.

THE CONSTRUCTION OF THE McKINLEY BRIDGE, at St. Louis, Mo., described on pp. 9-16 of this issue, will be held up for some time on account of the wrecking of the falsework under the second span by the huge ice-jamb in the Mississippi during the past week. As a result of the continued cold of the last part of December a large ice pack formed in the river above the city and practically closed the port to all navigation. Nevertheless, work on the construction of the timber centering for the second span of the McKinley Bridge has been going on daily and had nearly reached completion on Dec. 31, when the floe began to move. Warning was immediately given to the hundred or more workmen upon the structure and they all managed to clamber onto the adjacent spans or across the ice to a neighboring barge, before the moving ice had sheared the timber posts from their pile footings and tumbled the entire framework, some 400 ft. long and 100 ft. high, into the river. All the men engaged on the work were rescued but a considerable amount of erecting machinery as well as the timber of the falsework is a loss. No steelwork had been started on the span affected, and the falsework under the completed spans had been removed before the present ice formed.

The jamb continued to move down the river to the great menace of the shipping and wharves. At present, however, no very great damage has resulted.

AN ORDINANCE TO REGULATE THE USE OF water in sprinkling or flushing brick-paved streets in Wheeling, W. Va., provides:

Sec. 1. That it shall be unlawful for any person to use a hose in sprinkling or throwing water upon the streets paved with brick in the city of Wheeling, without having attached to the nozzle upon such hose or otherwise attached to such hose a sprinkler, so that the water passing through such hose shall be distributed upon the street instead of being thrown from the hose in one volume.

Sec. 2. Any person violating the provisions of this ordinance shall forfeit and pay to the city of Wheeling a fine of not less than $1 or more than $5, in the discretion of the police court of the city of Wheeling, and shall also pay the costs of the prosecution.

THE CIRCUMCONTINENTAL RAILWAY IN AUSTralia, connecting the existing lines with western terminus at Port Augusta in South Australia to those with eastern terminus at Kalgoorlie in Western Australia, described in Engineering News, Jan. 5, 1905, and July 30, 1908, has been reported on by the Consulting Engineer, Mr. H. Deane, and the report presented to the federal House of Representatives. The survey, just completed, shows that the new line would be about 1,063 miles long. A gage of 4 ft. 8½ ins. and a ruling grade of 1.2% have been assumed. As was supposed before the survey was started the greatest difficulty in the new line will be the water supply, needed both for the running of the road and to irrigate the land traversed. To remedy this the project includes a number of reservoirs and irrigation canals. The estimates made in the report is $17,000,000 for construction and one year's maintenance. No definite action on the report has yet been taken.

Personals.

Mr. Geo. D. Locke has become Vice-President and General Manager of the Arkansas, Oklahoma & Western R. R., with headquarters at Rogers, Ark.

Mr. F. D. Chase has resigned his position as Architect for the Illinois Central R. R. to accept a similar position with the Western Electric Co. at Hawthorne, Ill.

Mr. J. E. O'Brien, Mechanical Engineer of the Northern Pacific Ry., at St. Paul, Minn., has been appointed Superintendent of Motive Power of the Western Pacific Ry.

Mr. Oscar G. Murray, President of the Baltimore & Ohio R. R., has resigned, effective Jan. 14. Mr. Daniel Willard has been elected to succeed Mr. Murray, effective Jan. 15.

Mr. Alfred Noble, Past-President Am. Soc. C. E., relinquished his position as Chief Engineer of the East River division of the Pennsylvania Tunnel & Terminal R. R. on Dec. 31.

Capt. Benjamin Tappan, U. S. Navy, formerly in command of the battleship "Rhode Island," has become captain of the Mare Island Navy Yard, California, succeeding Capt. Lucien Young.

Dr. Henry M. Payne, Secretary of the Appalachian Engineering Association, has severed his connection with West Virginia University as Professor of Mining Engineering to become General Field Manager for the Hydraulic Mining Cartridge Co., of New York City.

Mr. Robert F. Kilpatrick, Assistant Superintendent of the motive power and car department of the Denver & Rio Grande R. R., has been appointed Master Mechanic of the Western Pacific Ry. at Stockton, Cal. Mr. Kilpatrick was formerly Superintendent of Motive Power of the Delaware, Lackawanna & Western R. R.

Mr. Frederic H. Keyes, M. Am. Soc. M. E., formerly General Manager of the Robb-Munford Boiler Co., has

associated himself with Mr. Timothy W. Sprague, Mr. Henry Docker Jackson and others to carry on a general consulting engineering practice at 88 Broad St., Boston, Mass., under the name of Timothy W. Sprague, Frederic H. Keyes, Henry D. Jackson & Associates.

Mr. W. F. Steffens, Assoc. M. Am. Soc. C. E., has resigned his position as Engineer of Bridges and Buildings of the Carolina, Clinchfield & Ohio Ry. to become Engineer of Structures of the Boston & Albany R. R., with headquarters at Boston, Mass. Mr. Ward Crosby, Division Engineer of Construction of the Carolina, Clinchfield & Ohio Ry., has been appointed Principal Assistant Engineer, with headquarters at Johnson City, Tenn., and the position of engineer of bridges and buildings, formerly held by Mr. Steffens, has been abolished.

Mr. John A. Vogelson, M. Am. Soc. C. E., has been appointed Chief of the Bureau of Health of Philadelphia, Pa., at a salary of $4,000 a year. Mr. Vogelson was employed in Philadelphia in 1901-5 in connection with the filtration system. In 1905 he spent some time in the Philippine Islands as Assistant Engineer in charge of construction of new water-works at Manila. He was later employed in the New York water department but returned to Philadelphia in 1907. At the time of his latest appointment he was Engineer in charge of construction of filtration plants.

Mr. W. T. Tyler, General Superintendent of the second district of the St. Louis & San Francisco R. R., and Mr. J. E. Hutchison, General Superintendent of the first district, have exchanged places. Mr. H. H. Brown, formerly Superintendent at Neodesha, Kan., has been transferred to Fort Smith, Ark., as Superintendent of the central division. Mr. S. T. Cantrell, formerly Assistant Superintendent at Sapulpa, Okla., has succeeded Mr. Brown as Superintendent of the Kansas division at Neodesha. Mr. S. H. Charles, Acting Superintendent at Birmingham, Ala., has been appointed Superintendent of the Red River division at Francis, Okla., and will be succeeded at Birmingham by Mr. J. G. Lorton.

Mr. George M. Brill and Mr. Horace C. Gardner have formed a partnership under the name of Brill & Gardner, with offices at 1134 Marquette Bldg., Chicago, Ill., for the purpose of continuing the engineering and architectural practice heretofore conducted by Mr. Brill. Mr. Brill has specialized for the past twelve years in the design of complete manufacturing and power plants. Mr. Gardner has been Manager of the construction and mechanical departments of Swift & Co., of Chicago, for more than 20 years and has had charge in this capacity of the construction and installation of packing houses, industrial plants, ice-making and refrigerating plants and of the building and maintenance of railway equipment.

Mr. W. J. Best, for the past three years Chicago Sales Manager and Engineer for the Buckeye Engine Co., has taken a position with the Wheeler Condenser & Engineering Co., Carteret, N. J., with the title of Treasurer. In addition to his duties as Treasurer, he will have charge of part of the sales and engineering work. Before his connection with the Buckeye Engine Co., Mr. Best was for five years Sales Engineer with the Green Fuel Economizer Co., in their Chicago office. He was previously Erecting Engineer for E. F. Williams, in which capacity he put in many large steam power stations, including those of the St. Joseph (Mo.) Railway & Light Co., the Imperial Electric Light Co. of St. Louis, Mo., and the New Orleans Edison Co. For five years previous to this he was Eastern Engineer, located at New York, for the Buckeye Engine Co.

New York City appointments have been announced by Mayor William J. Gaynor as follows:

Mr. Henry S. Thompson has been appointed Commissioner of Water Supply, Gas and Electricity, with a salary of $7,500 a year. Mr. Thompson is a graduate of Princeton University, class of 1894. He was one of the organizers of the Thompson-Starrett Co., from which, however, he withdrew about five years ago. He was Superintendent of Buildings for the Borough of Manhattan in 1903 and was appointed Commissioner of Public Works of the same borough in May, 1907.

Mr. Calvin Tomkins, Assoc. Am. Soc. C. E., has been appointed Commissioner of Docks and Ferries, with a salary of $7,500 a year. He is a graduate of Cornell University, class of 1879, and has been engaged for some time in the manufacture of building materials.

Mr. Edward W. Bemis, for the past few years Superintendent of Water Works of Cleveland, Ohio, has been appointed First Deputy Water Commissioner with a salary of $6,000. Some of the results of his work in the Cleveland water department were recorded in our issue of Nov. 4, 1909.

Mr. Kingsley L. Martin, M. Am. Soc. C. E., has been appointed Bridge Commissioner, with a salary of $7,500. Mr. Martin was made Assistant Engineer of the Brooklyn Bridge in 1892 and Assistant Engineer of the Williamsburg Bridge in 1896. During the war with Spain, he served in the U. S. Navy. He was made Engineer in charge of the Williamsburg Bridge in 1904, and two years later was put in charge of both the Williamsburg and the Brooklyn Bridge. He became Chief Engineer of the Department of Bridges in 1908.

Mr. Rudolph P. Miller, M. Am. Soc. C. E., has been appointed by President George McAneny to be Superintendent of Buildings of the Borough of Manhattan. Mr. Miller is a graduate of the Columbia School of Mines. He entered the Bureau of Buildings of Manhattan as an inspector in 1895 and later became Chief Engineer of the bureau, remaining in this position about five years. Since June 1, 1906, he has been in private practice as a consulting engineer.

Obituary.

Joseph F. Mitchell, Secretary and Treasurer of the Penn Bridge Co., Beaver Falls, Pa., died Dec. 21.

Frank B. Smith, President of the Crucible Steel Co. of America, died Dec. 30 at his home in Sewickley, Pa.

Harvey A. Shipman, M. Am. Inst. M. E., died Jan. 2 from pneumonia at Denver, Colo. Mr. Shipman had been in charge of mining operations in the United States, in Australia and in South Africa.

James W. Friend, Vice-President of the Pressed Steel Car Co., died at his home in Pittsburg, Pa., Dec. 26, aged 64 years. Mr. Friend was Vice-President of the Western Steel Car & Foundry Co. and was one of the owners of the Clinton Iron & Steel Co.

Camille S. D'Invellieres, M. Am. Soc. C. E., Engineer in charge of construction for the Pennsylvania R. R. in Pennsylvania, died from pneumonia Jan. 2 at his home in Cresson, Pa. He was 58 years old. In 1878 he went to Brazil with the Collins expedition to build the Madeira & Mamores R. R. and was later made Chief Engineer of the expedition.

George Thompson, Superintendent Engineer of the Cunard Steamship Co., died Dec. 27 from pneumonia at his home in Liverpool, England, aged 42 years. Mr. Thompson was formerly Works Manager for the shipbuilding firm of Harland & Wolff, of Belfast, Ireland, and entered the service of the Cunard Line in 1903 as Assistant Superintendent. He became Superintendent Engineer in 1904.

John H. Lidgerwood, Vice-President and Treasurer of the Lidgerwood Mfg. Co., of New York City, died Jan. 1 at his home in Morristown, N. J., at the age of 80 years. He had been confined to his home with neuritis for about three months. Mr. Lidgerwood was born in New York, but had resided in Morristown since his 16th year. His early business experience was gained in the Speedwell Iron Works, operated by his father-in-law, Judge Stephen Vail.

Sir Edward Leader Williams, M. Inst. C. E., Consulting Engineer of the Manchester Ship Canal, died Jan. 1 in London, England. He was born in Worcester, England, in 1828 and was educated in private schools. In 1846 he was engaged as an engineer on the works of the Great Northern Ry. and has since been employed on harbor work at Shoreham and Dover, and on the improvement of the River Weaver and the Bridgewater Canal. He was Chief Engineer of the Manchester Ship Canal during its construction. He contributed various papers, published in the "Proceedings" of the British Institution of Civil Engineers, and was formerly a Vice-President of the Institution.

Jacobus Van der Hoek, M. Am. Soc. C. E., was killed about two weeks ago in a tunnel near Wilkesbarre, Pa. He and an assistant were inspecting the tunnel and were run down by a fast freight train. Mr. Van der Hoek was born in 1862 and graduated at the Polytechnical School at Delft, Netherlands, in 1883. In 1890 he became a draftsman and assistant engineer for the Lehigh Valley R. R. Two years later he was made Resident Engineer on construction of the main line of the Lehigh Valley from Buffalo to Sayre and in 1893 he began a term as Division Engineer of the Buffalo division, in charge of engineering only. In 1898 he was given charge also of maintenance of way in the Buffalo division and remained as Division Engineer up to 1909. At the time of his death he was Engineer of Construction for the Lehigh Coal & Navigation Co., with office at Landsford, Pa.

Engineering Societies.

COMING MEETINGS.

NEW ENGLAND WATER WORKS ASSOCIATION.
Jan. 12. Annual meeting at Boston, Mass. Secy., Willard Kent, 715 Tremont Temple, Boston, Mass.

MICHIGAN ENGINEERING SOCIETY.
Jan. 12-14. Annual meeting at Lansing, Mich. Secy., Alba L. Holmes, 574 Wealthy Ave., Grand Rapids, Mich.

INDIANA ENGINEERING SOCIETY.
Jan. 13-15. Annual convention at Indianapolis, Ind. Secy., Chas. Brossmann, Union Trust Bldg., Indianapolis, Ind.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
Jan. 18-20. Annual meeting at New York City. Secy., W. M. Mackay, P. O. Box 1818, New York City.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Jan. 19-20. Annual meeting at New York City. Secy., Chas. W. Hunt, 220 West 57th St., New York City.

AMERICAN SOCIETY OF INSPECTORS OF PLUMBING AND SANITARY ENGINEERS.
Jan. 20-22. Annual convention at Trenton, N. J. Secy., C. S. McCosker, Mobile, Ala.

CANADIAN SOCIETY OF CIVIL ENGINEERS.
Jan. 25-27. Annual meeting at Ottawa, Ont. Secy., C. H. McLeod, 413 Dorchester St. West, Montreal, Que.

ILLINOIS SOCIETY OF ENGINEERS AND SURVEYORS.
Jan. 26-28. Annual meeting at Cairo, Ill. Secy., E. E. R. Tratman, 1636 Monadnock Block, Chicago, Ill.

NATIONAL BRICK MANUFACTURERS' ASSOCIATION.
Feb. 7-12. Annual convention at Pittsburg, Pa. Secy., T. A. Randall, Indianapolis, Ind.

CONNECTICUT SOCIETY OF CIVIL ENGINEERS.
Feb. 8. Annual meeting at New Haven, Conn. Secy., J. Frederick Jackson, Box 1304, New Haven, Conn.

OHIO ENGINEERING SOCIETY.
Feb. 8-10. Annual meeting at Columbus, Ohio. Secy., C. J. Knisely, New Philadelphia, Ohio.

NEW ENGLAND ASSOCIATION OF GAS ENGINEERS.
Feb. 16. Annual meeting at Boston, Mass. Secy., N. W. Gifford, 26 Central Square, E. Boston, Mass.

IOWA ENGINEERING SOCIETY.
Feb. 16-17. Annual convention at Cedar Rapids, Iowa. Secy., A. H. Ford, Iowa City, Iowa.

NORTHWESTERN CEMENT PRODUCTS ASSOCIATION.
Feb. 18-21. Annual convention at Chicago, Ill. Chairman, notification committee, O. U. Miracle, Minneapolis, Minn.

NATIONAL ASSOCIATION OF CEMENT USERS.
Feb. 21-26. Annual convention at Chicago, Ill. Pres., Richard L. Humphrey, Harrison Bldg., Philadelphia, Pa.

AMERICAN SOCIETY OF ENGINEERING CONTRACTORS.
Feb. 24-26. Annual meeting at Chicago, Ill. Secy., D. J. Hauer, 721 Park Row Bldg., New York City.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.—The January meeting of the Institute, to be held Jan. 14 at the Engineering Societies' Building in New York City, will be under the auspices of the Railway Committee. Prof. W. S. Franklin and Mr. S. S. Seyfert, of Lehigh University, will present a paper "On the Space Economy of the Single-Phase Series Motor."

CENTRAL RAILWAY CLUB.—The annual meeting will be held at Buffalo, N. Y., Jan. 13. Mr. W. O. Thompson, Master Car Builder of the western division of the New York Central & Hudson River R. R., will present a paper entitled "Car Interchange: Its Past, Present and Future."

NEW ENGLAND WATER WORKS ASSOCIATION.—At the annual meeting in Boston, Mass., Jan. 12, three papers will be presented as follows: "Government Policy in Relation to Water Powers," Marshall O. Leighton, Washington, D. C.; "The Maidstone Typhoid Epidemic," Wm. T. Mason, Troy, N. Y.; report of committee appointed to prepare "A Standard Specification for Fire Hydrants."

NORTHWESTERN CEMENT PRODUCTS ASSOCIATION.—On account of the illness of the President, Mr. Martin T. Roche, who has been confined to his home for the past two months, and on account of the resignation of the Secretary, Mr. A. E. Pfiffner, the sixth annual convention will not be held in St. Paul, Minn., March 1-5, as formerly planned. It has been arranged to hold the convention in Chicago, Ill., Feb. 18-21 in conjunction with the cement show at the Coliseum.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.—Meetings will be held in New York City on Jan. 11, in St. Louis on Jan. 15, and in Boston on Jan. 21. The spring meeting of the society will be held this year as usual, in addition to the London meeting which occurs in July, and will take place at Atlantic City, N. J., May 31-June 3. The headquarters during the meeting will be at the Hotel Marlboro-Blenheim.

The January meeting in Boston will take the form of a banquet at the Hotel Somerset to be tendered jointly by The American Society of Mechanical Engineers, the Boston Society of Civil Engineers and the Boston branch of the American Institute of Electrical Engineers, to the presidents of the three societies, George Westinghouse, Geo. B. Francis and L. B. Stillwell, respectively, and other guests, including John Anderson Bensel, the incoming president of the American Society of Civil Engineers. Following the banquet there will be addresses by several of the guests and a paper will be presented on "The Main and Auxiliary Machinery of the Battleship "North Dakota," illustrated by lantern slides. The president of the Boston Society of Civil Engineers will outline what his society has accomplished toward a project that has been under discussion at Boston for a united engineering building to be occupied by the several professional engineering organizations located in the city and vicinity.

At the New York meeting, Jan. 11, a paper by Professor F. H. Sibley, of the University of Alabama, will be presented on "Efficiency Tests of Lubricating Oils." Contributions on the characteristics of lubricants will be presented by Dr. C. F. Mabery, of Case School of Applied Science, Cleveland, Ohio, and Mr. Chas. Miller, of Franklin, Pa. Among those who will discuss the paper are Mr. F. R. Low and Mr. J. P. Sparrow, of New York City and Mr. I. E. Moultrop, of Boston, Mass.